VOLUME 84 NO. SA3

JUNE 1958

JOURNAL of the

Sanitary Engineering Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

AMERICAN SOCIETY OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, ⅛-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller in the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper within a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be ⅛-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class mail privileges are authorized at Ann Arbor, Michigan.

HY,PO,SA,ST.

Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

June, 1958

Papers

(Over)

Number

Paper 1665 June, 1958 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SEWAGE PUMPING

H. H. Benjes,[1] M. ASCE
(Proc. Paper 1665)

General Considerations

Location, arrangement, type of equipment and structure, and external appearance are all basic considerations in the design of any sewage pumping station. Of primary consideration is the proper location of the station, which requires that a comprehensive study be made of the drainage area served to assure that the entire area can be adequately drained. Careful attention should be given, especially in undeveloped or partially developed areas, to probable future growth, because the location of the pumping station will in many cases determine the future overall development of the area.

Studies of the topography of the land downstream from a proposed station should be made to determine if the station can be eliminated by tunneling through high ground or by providing conduits around high ground to obtain gravity drainage. All pumping stations require operation and maintenance, the cost of which when capitalized will usually justify a considerable first cost expenditure to obtain gravity drainage.

The capacities of downstream conduits sometimes limit the maximum station capacity and should always be investigated.

The depth of incoming sewers will usually determine the depth of the station substructure below the ground level. Surface conditions, environment and relative elevations with reference to flooding will generally fix the level of the operating floor, the type of superstructure and degree of exterior finish and trim.

Where stations are located in built-up areas, the station superstructure should be similar to adjacent structures. Where isolated, the type of structure can be left to the discretion of the owner and engineer.

In most sewage pumping stations, dependability of equipment and power is mandatory because failure of a station can cause considerable damage. For this reason, types of drives for pumping equipment should be given careful consideration. To be considered reliable, electric stations should have at

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1665 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

1. Dept. Head, Civ. Eng., Black & Veatch, Cons. Engrs., Kansas City, Mo.

least two incoming power lines with automatic switching equipment to transfer the load from the preferred power line to the standby power line in the event of a power failure in the preferred line. Where this reliability cannot be obtained, standby engine driven generators, standby engine driven pumping units, or all engine driven pumping units may be required.

In many instances, sewage pumping stations can be located so that the sewage may be overflowed in the event of a power failure or in the event the station pumping capacity is exceeded. Such overflows can be permitted, however, only in locations where a temporary overflow of sewage will not be hazardous to health or injurious to property.

Careful consideration should be given to the control of odors, especially when stations are located within 1000 feet of human habitation. This may be done to a large extent by preventing deposition of solids in the wet well and by adequate ventilation. However, when stale sewage is being handled, it is sometimes impossible to prevent odors emanating from the station without special treatment of the ventilating air.

Station Capacity

The establishment of the station capacity is dependent upon studies of development in the area tributary to the station and forecasts of probable future growth. If the area is not fully developed, initial station capacity should be provided to meet the probable requirements for a reasonable time in the future, customarily for a period of not less than 10 years. Since the initial flows may not be as great as allowed in the design, the effects of the minimum flow conditions must be considered to assure that retention of sewage in the wet well will not create a nuisance and that the pumping equipment will not operate too infrequently. Future requirements for station capacity must also be given consideration to allow for expansion as required to meet the inflow conditions as they develop.

Probable minimum inflows will affect the design of screen flow-through channels and the wet well size. Average flow conditions are of interest in that they indicate the conditions under which the station will usually operate. Pumping equipment should be selected to perform at maximum efficiencies under the average conditions to obtain the least operating expense.

Wet Well Design

Wet well storage capacity is required in all sewage pumping stations, since it is impossible to match pumping rates exactly with inflow rates to a station. The selection of the proper storage capacity for the wet well is critical because it affects (1) the time which the liquid will be retained in the station, and (2) the frequency of operation of the pumping equipment.

Although it is desirable to balance wet well capacity and pumping rates so as to approximate as closely as possible the rate of inflow to the wet well, this desirable condition is seldom realized. While from the mechanical viewpoint it is desirable to operate a pump continuously, or for long periods of time, such performance is not compatible with the maintenance of aerobic conditions in the sewage if it results in long retention periods.

The shape of the wet well and the detention provided, should be such that deposition is minimized and the sewage does not become septic. While design

policies usually base detention upon the average rate of flow, the maximum and minimum rates will be the determining factors in sizing the wet well. Most stations will accomplish the desired results with a minimum of objections, if the size of the wet well is such that with any combination of inflow and pumping, the cycle of operation for each pump will be not less than 5 minutes, and the maximum retention time in the wet well will not exceed 30 minutes. To accomplish these requirements, the design of the wet well must be coordinated with the selection of the individual pumping units and the selection of the sump levels at which the pumps are started and stopped. If the wet well capacity is determined as outlined above, the pumps, electric motors, and electrical controls will function in a satisfactory manner and sewage will seldom be retained in the wet well for too long a period of time.

It should be kept in mind that the longer the detention of the sewage in the wet well, the greater the chance for objectionable odors and the greater the possibility of deposition. Therefore, detention periods should be kept to a minimum compatible with proper operation of the pumping equipment.

The pump control levels in the wet well should be such that the incoming sewers to the station will not be surcharged and such that the pumps will be submerged when starting.

Probably the most controversial point in the design of wet wells is the bottom slope needed to minimize deposition of solids. It has been found that solids will not accumulate on slopes greater than 1 horizontal to 1.75 vertical.

Bell mouth suction inlets are superior to inlets with sharp edges because there is less tendency for material to accumulate and also because the flow is brought into the suction pipe in a more uniform manner with less head loss. The spacing between suction inlets in the wet well should prevent hydraulic interference and deposition of solids, insofar as possible. A spacing of not more than two times the suction pipe diameter is required to completely prevent deposition of solids between the suction inlets. This requirement cannot be satisfied, since a spacing of approximately four times the diameter of the suction inlets is required to prevent hydraulic interference.

Provision should be made to provide adequate access to wet wells for inspection and cleaning. In the larger stations stairways and walkway platforms should be provided so that the wet well may be squeegeed and hosed down. Any ladders should be provided with safety cages. Proper lighting and ventilating facilities should also be readily available when the wet well is entered for any reason.

Where wet wells are totally enclosed, adequate vents should be provided to allow for entrance and exit of air as the liquid level in the sump rises and falls. Rooms located directly over wet wells should be isolated from the remainder of the station and provided with ventilating facilities which blow air into the rooms to provide a pressure slightly above atmospheric.

Screening

It is customary to protect pumping equipment with bar screens or comminutors to prevent large objects from entering the wet well. Bar screens may be of the manually cleaned or mechanically cleaned type. Manually cleaned screens are usually of the basket or bar rack type. Basket screens are applicable only to smaller pumping stations and generally are fabricated in the form of a box with one open side facing the incoming sewer or drain

line. Screenings are collected in the basket and facilities are provided for hoisting the basket to the surface where the screenings may be removed. Manually cleaned bar screens are placed in the entrance channel at angles varying between 30 and 45 degrees with the horizontal. Due to the difficulty of cleaning by hand, manually cleaned bar screens are applicable only to channels of relatively shallow depths.

Mechanically cleaned bar screens are basically of the front cleaned or rear cleaned type, with the bar screen making an angle with the horizontal, varying from 60 to 90 degrees. The front cleaned type bar screen has the advantage that it can be placed in a channel of almost unlimited depth. However, since the cleaning mechanism is on the upstream side of the bar screen, it has the disadvantage that objects may wedge between the cleaning mechanism and the bottom of the channel so that the machine is disabled. This disadvantage is overcome by the back cleaned bar screen where all of the cleaning mechanism is behind the bar screen. The back cleaned bar screen does have the disadvantage that it can be installed only in channels with limited depths of flow, since the individual bars in the bar screen must be supported from the bottom of the channel.

Bar screens are usually fabricated of iron or steel bars placed in a vertical direction. The space between the individual bars varies. A clear space of 1-1/2 to 2-1/2 inches is usually used for manually cleaned screens. A 1-inch spacing is customary for the mechanical type.

Bar screens should be sized so that the velocity through the clear openings of the screen will not exceed 2-1/2 feet per second under any flow condition. Where mechanically cleaned bar screens are used, a 1-foot minimum depth of flow should be maintained ahead of the screens to assure satisfactory operation of the cleaning mechanism.

A head loss of at least 6 inches should be allowed in the hydraulics of a station for loss of head through the bar screens. Where manually cleaned screens are used, especially where they will receive relatively little attention, a greater head loss should be allowed. The floor of the screen channel should be placed at least 6 inches below the invert of the incoming sewers to allow for some accumulation of screenings without affecting the flow in the sewers.

Screenings should be disposed of by burying, burning, or by shredding; in the latter case the screenings may be returned to the wet well of the station. Mechanical shredders should be adequately sized to handle the screenings which will be removed. The handling and disposal of screenings are important design features. Inadequate facilities may result in a nuisance.

A water supply should be provided, if possible, at any screening installation in order that cleanliness may be maintained. Shredders for screenings require a water supply, and care should be exercised in the selection of shredding equipment to assure that there is no possibility of a cross-connection.

Pumping Equipment

Types in General Use and Applicability

Pumping equipment used in sewage stations may be classified into two general types: centrifugal pumps, and pneumatic ejectors. Pneumatic ejectors are used only in the smaller installations.

Centrifugal Pumps

Centrifugal pumps fall into three general classifications, as follows:

(a) Axial flow or propeller pumps
(b) Mixed flow or angle flow pumps
(c) Radial flow pumps (commonly referred to as centrifugal pumps).

Mixed flow and single suction radial flow pumps are the types most commonly used for sewage pumping.

A comparison of the relative characteristics of axial flow, mixed flow, and centrifugal pumps is shown in Table I.

Impeller Types

Impellers for centrifugal pumps may be classified into three general types:

(a) Enclosed
(b) Open
(c) Semi-enclosed.

Enclosed impellers with shrouds are generally specified for pumps which handle sanitary sewage since they are less subject to clogging and also are better able to pass stringy materials than other types.

So-called "non-clog pumps" are all based on an original development by Wood at New Orleans. Actually, no pump has been developed that cannot clog, either in the pump or its appurtenances. Experience shows that rope, long stringy rags, sticks, cans, rubber goods and grease are objects most inducive to clogging.

Non-clog pumps are used almost exclusively today for pumps smaller than 10 inches in size for the handling of sanitary sewage. These pumps differ from conventional pumps in arrangement, size, smoothness and contour of channels and impellers to permit passage of clogging material through the pump. The pump casing is a simple volute with a so-called end suction inlet to the impeller eye. The conventional non-clog impeller contains two blades, although some manufacturers are now offering a single blade (so-called "bladeless") impeller. Impeller blades have smooth easy curves and are designed to prevent solids from collecting around the shaft between the impeller and backhead or casing.

Pneumatic Ejectors

Pneumatic ejectors are largely applicable for lifting sewage from the basements of buildings and for small lift stations where their advantages offset their low efficiency, which is limited to about 15 per cent. These advantages are: (1) sewage pumps are completely enclosed and consequently no sewer gas can escape except through the vent; (2) operation is fully automatic and the ejector goes into service only when needed; (3) the relatively few moving parts in contact with sewage require little attention or lubrication; (4) ejectors are not easily clogged; and (5) screening is not required as check valves and connecting lines will pass all solids that enter the ejector compartment.

Pump Selection

The number of pumps to be installed in the station will be dependent largely on the station capacity and range of flow. In considering station capacity, it is

customary to provide a total pumping capacity equal to the maximum expected inflow with at least one of the largest pumping units out of service. In large stations, two units are sometimes considered out of service in determining firm capacity. A minimum of two pumps should be installed in any station except where pneumatic ejectors are installed to serve less than 50 houses.

In small stations with maximum inflows of less than 1.0 mgd, two pumps only are customarily installed, with each unit having capacity sufficient to meet the maximum inflow rate. For larger stations, the size and number of units should be selected so that the range of inflow can be met without starting and stopping pumps too frequently and without requiring excessive wet well storage capacity. In many cases variable speed drives are provided in order to match the pumping rate with the inflow rate.

Many times, the capacity and depth of the wet well can be coordinated with the pumping units so that the rise and fall of the water level in the wet well will result in a variable pumping capacity which will nearly match the inflow rates. This is possible since all centrifugal pumps have the inherent characteristic that as the head increases the capacity decreases. In stations where the pumping head is low, the normal range of levels in the wet well will vary the pumping head so that a wide range of capacities may be obtained with each individual unit.

Before describing the mechanics of actually selecting pumping equipment, a clear understanding should be reached as to the meanings of various standard terms used in connection with describing pump characteristics. These terms and their definitions would include the following, as given by the Standards of the Hydraulic Institute.(1)

(a) Datum. All readings for suction lift, suction head and total discharge head, are taken with reference to datum which, in the case of horizontal shaft pumps, is the elevation of the pump centerline and in the case of vertical shaft pumps is the elevation of the entrance eye of the suction impeller.

(b) Suction Lift (h_s). Suction lift exists where the total suction head is below atmospheric pressure. Total suction lift, as determined on test, is the reading of a liquid manometer at the suction nozzle of the pump, converted to feet of liquid, and referred to datum, minus the velocity head at the point of gauge attachment.

(c) Suction Head (h_s). Suction head exists when the total suction head is above atmospheric pressure. Total suction head, as determined on test, is the reading of the gauge at the suction of the pump converted to feet of liquid and referred to datum, plus the velocity head at the point of gauge attachment.

(d) Total Discharge Head (h_d). Total discharge head is the reading of a pressure gauge at the discharge of the pump, converted to feet of liquid and referred to datum, plus velocity head at the point of gauge attachment.

(e) Total Head (H), (sometimes referred to as Total Dynamic Head or TDH). Total head is the measure of the energy increase per pound of the liquid imparted to it by the pump and is therefore the algebraic difference between the total discharge head and the total suction head. Total head, as determined on test where suction lift exists, is the sum of the total discharge head and total suction lift, and where positive

suction head exists, the total head is the total discharge head minus the total suction head.

Graphic representations of pump head relationships are shown in Figs. 1 and 2.

Pumps should be selected having head-capacity characteristics which correspond as nearly as possible to the overall station requirements. This can best be accomplished by the preparation of a system head-capacity curve showing all conditions of head and capacity under which the pumps will be required to operate. The head-capacity curve is developed using standard hydraulic methods for determining friction losses. There are a number of methods available for calculating pipe friction losses which are described in most standard hydraulic text books. Friction losses in straight piping are usually calculated using the Williams-Hazen formula. When long pipe lines are involved, it is impossible to predict accurately the total friction loss to be realized over an extended period of time. When the line is new, friction losses will be at a minimum and will probably increase with use. These friction losses will materially affect the capacity of the pumping units and also their successful operation. For this reason, system curves should be developed to show the possible maximum and minimum friction losses in the pipe line to be expected during the life time of the pumping units.

Where two or more pumps discharging into a common header and pipe line are being considered, it is usually advantageous to omit the head losses in the individual pump suction and discharge lines from the system head-capacity curves. This is advisable because the pumping capacity of each unit will vary depending upon which units are operating in parallel. To obtain a true picture of the output capacity of multiple pump operation, it is better to deduct the head losses in the suction and discharge piping for each individual pump from the individual pump characteristic curve. This provides a modified curve which is the pump performance at the station header. By adding the capacities for points of equal head, as shown by the modified curves, performance of multiple pump operation at the station discharge header is obtained. Fig. 3 shows a typical set of system curves, together with representative individual pump characteristic curves, modified pump curves, and combined modified pump curves showing multiple pump operation. The intersection of the modified pump curves and the combined modified pump curves with the system head capacity curves shows the station pumping capacity for the several conditions of operation. Fig. 3 shows four system curves, two curves with a Williams-Hazen coefficient of $C = 100$ for maximum and minimum water levels in the sump and two curves with a Williams-Hazen coefficient of $C = 140$ for maximum and minimum water levels. These coefficients usually can be considered as the maximum and minimum coefficients which will be obtained in sewage force mains.

It is good practice to select pumps that will deliver the station capacity at the maximum head. This capacity and head will not necessarily be the point of maximum pump efficiency. Pumps should be selected having their maximum efficiency at the average operating conditions. In the case of Fig. 3, assuming that the total station capacity is to be obtained by operating Pumps Nos. 1, 2 and 3 in parallel, the total head required at the station discharge header would be approximately 51 feet, with maximum sump level and assuming $C = 100$ in the discharge line. Projecting this point horizontally to the individual modified pump curves and thence vertically to the pump characteristic curves, the required head for Pumps Nos. 1 and 2 would be 54 feet and

for Pump No. 3 approximately 57 feet. The difference in head between the head required at the station header and the head required for each pump is the head loss in the suction and discharge piping for each individual pump.

Fig. 3 also indicates the minimum head at which each individual pump may operate. In the case of Pumps Nos. 1 and 2 this minimum head is approximately 39 feet. For Pump No. 3 it is approximately 42 feet. These minimum heads are important and should be furnished to the pump manufacturer since they will usually determine the maximum brake horsepower required to drive the pump and the maximum speed at which the pump may operate without cavitation. When purchasing pumps, the manufacturer of the pumping equipment should be given the minimum head against which the equipment is to operate in order that he may make a proper selection of the equipment to be furnished.

It must be remembered that the capacity of a centrifugal pump is a variable and will depend upon the total head at which the unit operates. Capacities for multiple pump operation can be obtained only by a study of the individual pump curves and system curves similar to Fig. 3. When a pump is referred to as having a certain capacity, this capacity applies only to one point on the characteristic curve and will vary depending upon the actual pumping head conditions.

The maximum speed at which a pump should operate is determined by the net positive suction head available at the pump, the quantity of liquid being pumped, and the total head. When specifying pumps, especially those which are to operate with a suction lift, the speed at which the pumps will operate should be checked against the Hydraulic Institute[1] limiting suction requirements.

In general, it is not good practice to operate sewage pumping units at speeds in excess of 1750 rpm. This speed is applicable only to smaller units. Larger units should operate at a slower speed.

Sewage pumps should be of the dry pit type, as wet pit pumps are difficult to maintain because they are not accessible to the operator except by pulling the pump. At most station locations, pulling a wet pit pump creates a nuisance and a health hazard.

Sewage pumps are available in both horizontal and vertical units. Most installations are now being made with the vertical type unit since they require less space than the horizontal, and where the pump pit is subject to the possibility of flooding, the pump drives may be mounted above possible flooding. The determination of whether or not to use vertical or horizontal units must be decided by the individual conditions and no general rule can be established as to which is preferable. The horizontal units are basically more stable than close coupled vertical units since the motors for the former are mounted separately from the pumps. Extended shafting for vertical units presents problems of alignment and maintenance, especially in larger installations, and must be weighed against the disadvantage of horizontal units with the drives mounted in the pump pit and consequently subject to flooding.

Pump Drives

In modern sewage pumping stations, pumps are driven either by electric motors or internal combustion engines. Where firm power and continuous duty are anticipated, electric motor driven units usually prove to be most economical in first cost and in maintenance. Where firm power is not available or where the duty is such that the pumping will be at infrequent intervals, gasoline, diesel or gas engines may prove to be the most economical and reliable.

There is no shortcut to the selection of the proper drive to apply to sewage pumping equipment. The proper drive can be selected only after the engineer has made a careful study of all factors concerned. The factors to be considered in the selection should at least include:

(a) First cost
(b) Availability and dependability of electric power
(c) Characteristics as to horsepower, torque and speed under all operating conditions
(d) Total annual energy cost considering not only the energy rate but demand charges
(e) Maintenance costs
(f) Dependability of equipment.

Piping and Valves

Piping usually is sized such that the maximum velocity in the suction piping and in the discharge piping will not exceed 5 fps and 8 fps respectively. Piping less than 4 inches in size should not be used for conveying sewage. Valves should be provided on the suction and discharge side of each pumping unit to allow proper maintenance of the pump.

Increasers and decreasers should be placed on each side of pumps to prevent velocities in excess of those mentioned above through gate valves and swing check valves. Pump discharge piping should not enter the header piping from the bottom since solids will have a tendency to settle out from the header into any vertical riser. Suction piping should be so arranged that it can be readily dismantled for cleaning. Each pumping unit must have a separate suction line from the wet well for satisfactory operation.

In laying out flanged piping it is essential that flexibility be provided by the use of hub end joints or other means, to the extent that tolerances in flange piping may be taken up in the flexible joints. Cast iron flanged pipe, specials, or fittings should never be encased in concrete since a failure in the flange would be very difficult to repair and since rigid connections should not be made between flanged piping and concrete walls or floors.

The proper hanging, bracing and support of all piping to assure that no undue strains are induced is essential. Particular attention must be paid to proper blocking and tying of pipe where hub end or flexible joints are used in piping runs. Small drain valves and air relief valves should be installed at all low and high points in the piping respectively to allow draining of piping and for the release of air.

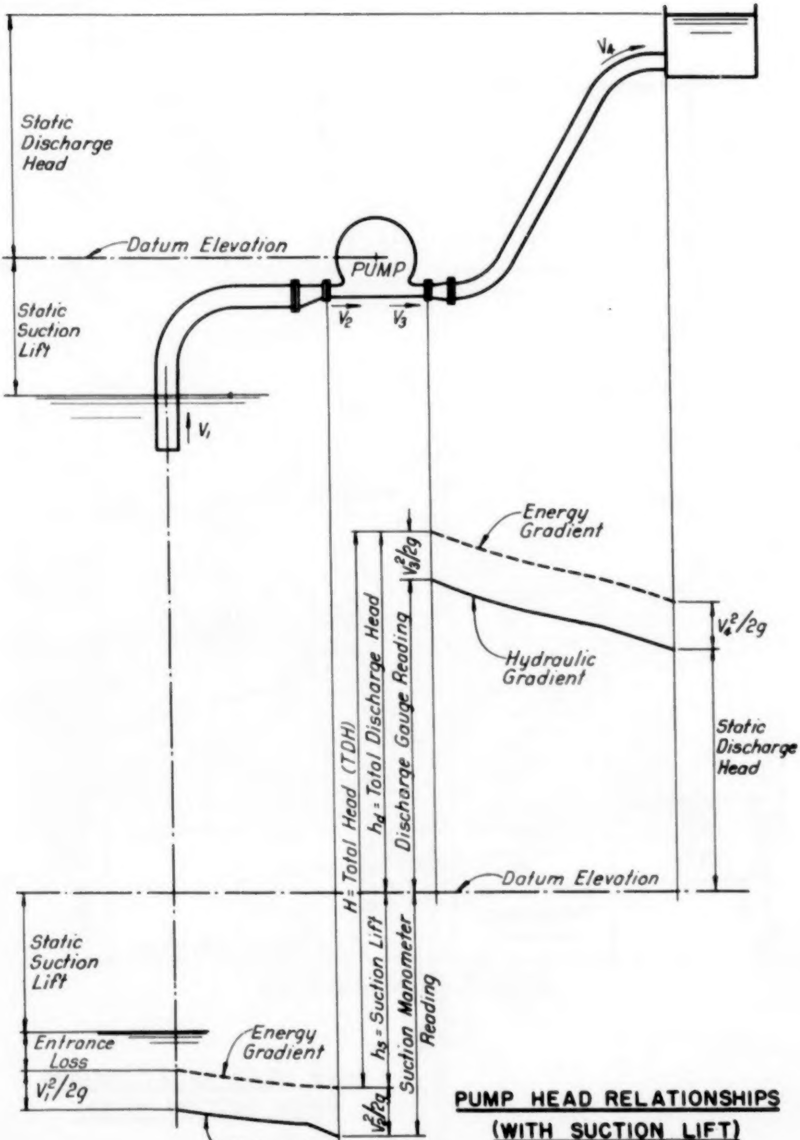
REFERENCES

1. Standards of Hydraulic Institute (Ninth Edition) Hydraulic Institute, New York 17, New York, (1951).

TABLE I

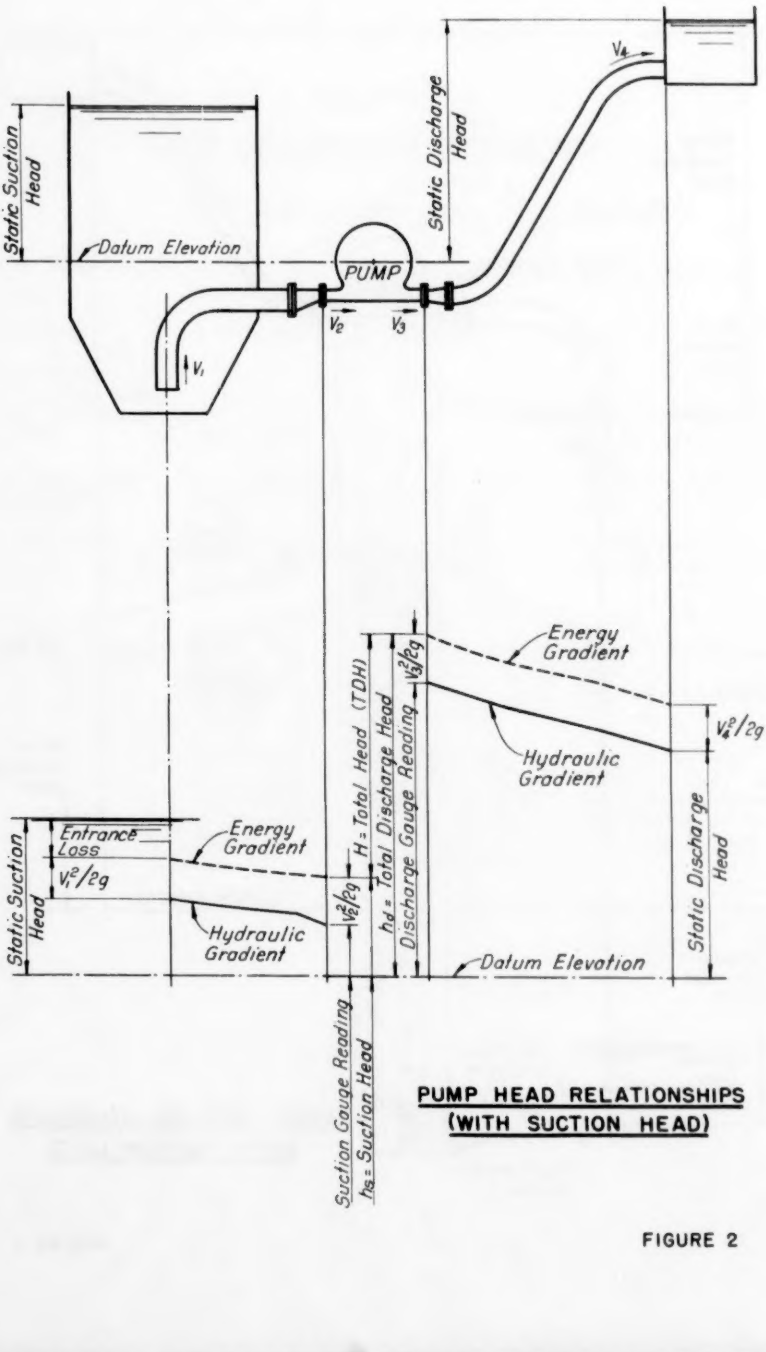
RELATIVE CHARACTERISTICS OF CENTRIFUGAL PUMPS

	Axial Flow	Mixed Flow	Radial Flow
Usual capacity range	Above 10,000 gpm	Above 5,000 gpm	Any
Head range	0 to 40 feet	25-100 feet	Any
Shut-off head above rated head (max. efficiency point)	About 200%	160%	120 to 140%
Horsepower characteristic	Decreases with capacity	Flat	Increases with capacity
Suction lift	Usually requires submergence	Usually requires submergence (lift limited)	Usually not over 15 feet
Specific speed	8,000 to 16,000	4,200 to 9,000	Below 420 - single suction Below 6000 - double suction
Service	Used where space and cost are considerations and load factor is low	Used where load factor is high and for service where trash and other solid material are encountered	Used where load factor is high and high efficiency and ease of maintenance are desired



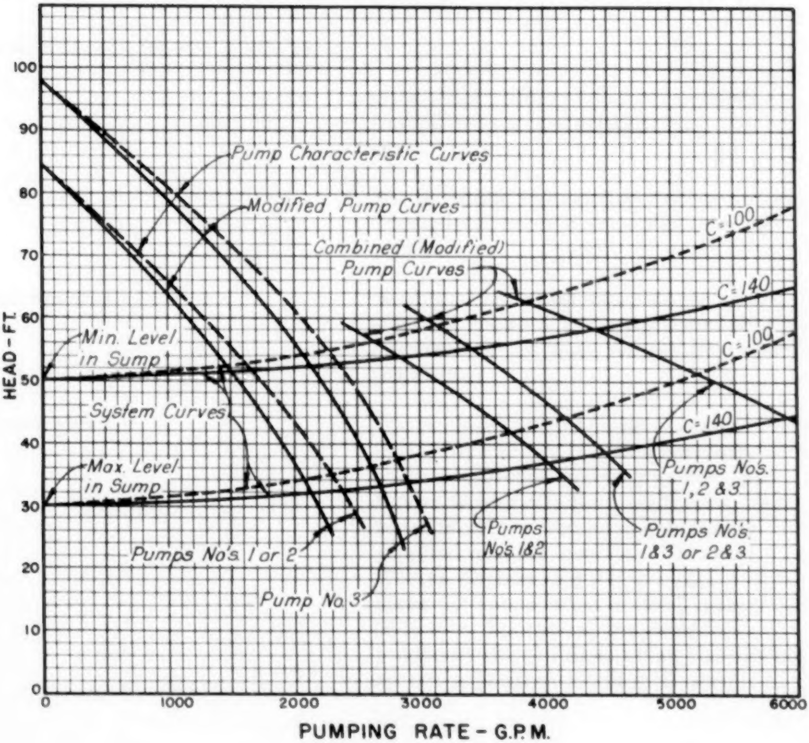
PUMP HEAD RELATIONSHIPS
(WITH SUCTION LIFT)

FIGURE 1



PUMP HEAD RELATIONSHIPS
(WITH SUCTION HEAD)

FIGURE 2



HEAD-CAPACITY CURVES

FIGURE 3

Paper 1677 June, 1958 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SED RESEARCH REPORT NO. 18

On	MUNICIPAL INCINERATOR DESIGN A SURVEY OF ENGINEERING PRACTICE
By	The Sanitary Engineering Research Committee, Solid Wastes Engineering Section
From Research Data of	Municipal and other officials who have supplied answers to a questionnaire prepared by the Solid Wastes Engineering Section
Acknowledgment	The Sanitary Engineering Division gratefully recognizes the generosity and professional courtesy of the many Public Works officials, engineers, and administrators who have generously supplied data for review. The Solid Wastes Engineering Section also recognizes the cooperation of the Department of Health, Education, and Welfare, Public Health Service, which has assisted in the preparation of this report.

SYNOPSIS

The results of detailed information supplied by numerous Public Works officials concerning the operation of their municipal incinerators is presented. Over 110 replies from various cities in the United States were received out of a total of 230 cities reportedly using incinerators as a method of solid wastes disposal. A comprehensive evaluation of the construction trends and costs is presented. In addition, detailed information on the type of refuse incinerated, the design data for the various incinerator components, and also other appropriate operational factors, including costs, are summarized and reviewed.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary ASCE. Paper 1677 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

INTRODUCTION

Recently questions have arisen concerning the adequacy of the basic incinerator plant design criteria available to meet today's municipal refuse disposal requirements. The Solid Wastes Engineering Section conducted this study to enable better evaluation of conditions in the refuse incinerator engineering field.

In March, 1957, questionnaires were mailed to some 230 cities in the U. S. reportedly using incineration as a method of municipal refuse disposal. Replies were received from 110 cities of which number ten reported using a plant operated by another city, county, or authority, and thirteen reported having abandoned their plants. Comprehensive replies were received from eighty-eight cities. Additional data published in tabular form in the literature permitted compilation of questionnaire data sheets for fourteen additional cities. Thus, the information contained in this report is based on an analysis of data on the newest or best plants currently operating in 102 cities in twenty-eight states and the District of Columbia. The breakdown of these cities according to population group is listed in Table I.

Table I

Number of Cities Reporting

Population Groups	Number of Cities
1,000,000 and over	5
500,000 to 1,000,000	10
100,000 to 500,000	25
50,000 to 100,000	16
25,000 to 50,000	17
5,000 to 25,000	29
Total	102

Trends in Construction

Fig. 1 shows the year construction was completed on the newest or best incinerator plant in 100 cities (two cities did not report on this item). It is interesting to note that 11 (or 10%) of the plants were completed prior to 1931 and are over 25 years of age; 34 (or 34%) were constructed prior to 1946 and range from ten to 25 years old; and slightly more than half (or 55%) have been placed in operation during the past ten years.

It is of further interest that the 13 communities reporting abandonment of their plants did so since 1951 and eight reported the adoption of the sanitary landfill method. Ten of the 13 cities had less than 50,000 population. The dates of construction of the abandoned plants were reported in four instances; their ages when abandoned were 7, 16, 18, and 20 years respectively.

Fig. 2 shows a graphic presentation according to population of the dates of completion of construction of the newest or best plant currently operating in 86 cities. The trend toward increasing utilization of incineration as a method of refuse disposal in the larger cities is significantly demonstrated.

It should be kept in mind in interpreting this data that it represents the newest or best plant in each city. Data received from the 88 basic study cities showed 120 municipal incineration plants are in operation. It is estimated that about 145 plants are operated by the entire study group of 102 cities.

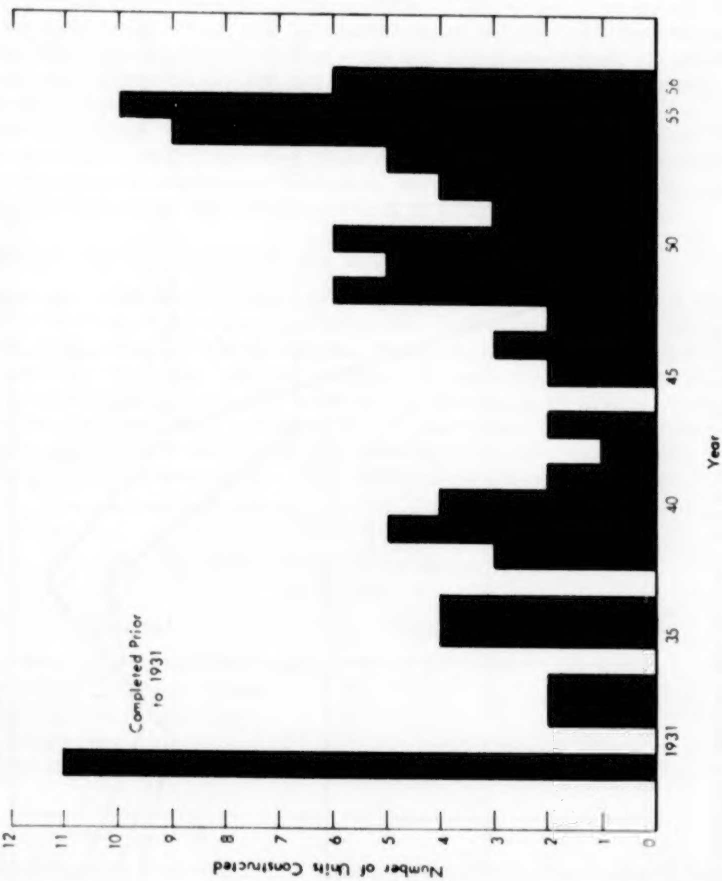
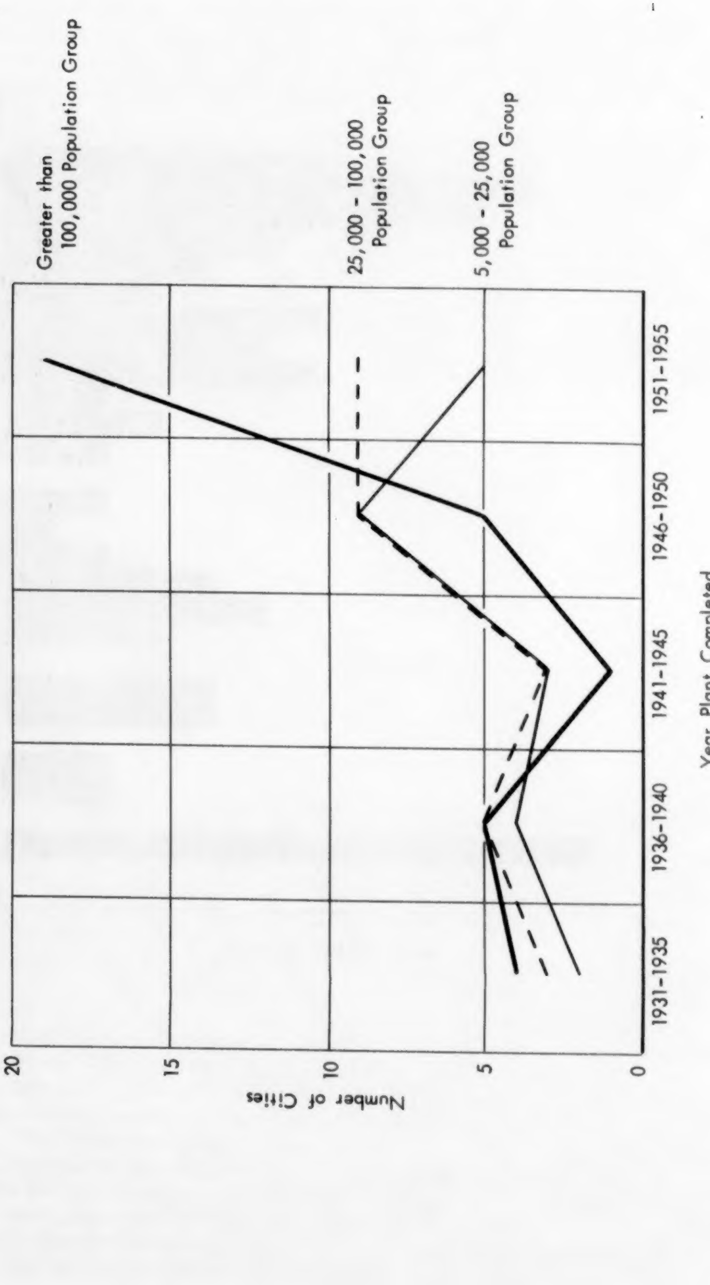


Figure 1 YEAR CONSTRUCTION COMPLETED ON 100 INCINERATOR PLANTS REPORTED OPERATION, 1957

Figure 2 INCINERATION TRENDS ACCORDING TO POPULATION (86 Cities)



Responsibility for Engineering Plant Construction

Eighty-eight cities provided information on where the responsibility for engineering the plants was centered. Consulting engineers had primary responsibility in 54 (or 62%) of the cities. The municipal engineering staffs of 14 (or 16%) of the cities bore the responsibility for the design; the manufacturer in 17 (or 19%) of the cities, and the manufacturer and the city engineering staffs took this responsibility jointly in three (or 3%) of the cities. Of significance is the fact that in 34 (or 39%) of the cases reported a consulting engineer was not involved in the design of the plant.

Seventy-nine of the 88 cities responded to the query "are you satisfied with the plant performance?" Sixty-nine reported, yes, ten, no. Of the ten plants reported giving unsatisfactory performance, seven were reported to have been designed and constructed without the services of a consulting engineer.

Type of Refuse Material Acceptable for Incineration

Garbage and combustible rubbish is the predominant classification of refuse accepted for burning at municipal incinerators. Combined or mixed refuse is the second most important classification. Together, these two classifications represented 87% of the type material accepted at municipal incinerator plants. Table II lists the types of material accepted for burning in 97 cities. Four of the cities reported accepting garbage only. It is probable that this garbage is at least wrapped in paper and likely that combustible or non-combustible food containers are also included in the local definition of the term "garbage."

Table II

Type of Refuse Accepted for Incineration
(97 cities)

Material	Number of Cities
1. Garbage and combustible rubbish	59
2. Combined or mixed refuse	25
3. Combustible rubbish only	9
4. Garbage only	4
5. Rubbish (combustible or non-combustible)	0

Disposal of Incinerator Ash

Eighty-five cities reported on the ultimate disposition of incinerator ash. Of this number 65 (or over 75%) dispose of incinerator ash in sanitary landfills, only 20 reported that they did not. Of significance is the fact that 20 of the cities use the sanitary landfill method solely for the disposal of incinerator ash. Forty-five reported the use of the sanitary landfill not only for incinerator ash but also as a method of refuse disposal along with incineration.

Weighing of Refuse to be Incinerated

Eighty-six cities reported on whether refuse to be burned was weighed. Forty-nine replied in the affirmative. Forty-three per cent (37) reported that refuse was not weighed.

Design Data

Grates

Fig. 3 is a graphical presentation of the type of grates used in 92 plants and the year they were placed in operation. The figure demonstrates the complete change in practice with respect to hand stoking with only one such plant being constructed during the past seven years. On the other hand reliance on automatic or semi-automatic grates is now apparently universally accepted practice. One rotary kiln type furnace was reported in operation (Atlanta, Georgia). A second was placed in operation in 1957 in Louisville, Kentucky. The travelling grate type furnace represents a new development in municipal incinerator design. The plant listed in the 1954-1956 bar graph of Fig. 3 represents New York City's new southside incinerator.

Grate Loading Factors

Grate loading design factors depend largely on the type of grate used and the type of refuse to be burned. Data on grate loading design factors were reported by 41 cities. These were listed in Table III according to type of refuse burned and type of grate used. Probably the outstanding characteristic of grate loading design evidenced by the data is their relatively wide variation in actual practice. For example, eight cities reported using Rotary Circular type furnaces for incinerating garbage and combustible rubbish. Grate loading factors reported varied from 25 to 120, or almost 500%. These plants were all constructed in the period 1949-1956. A somewhat similar relationship insofar as range is concerned exists in the case of plants with hand stoked type grates in their furnace or furnaces. In this instance, however, all the furnaces were constructed or installed prior to 1943 with one exception, a plant built in 1945. There were less data reported on plants using automatic or semi-automatic sloped grates. However, the range in grate design factors reportedly used in five plants designed to burn mixed refuse was about 50% (48-71 pounds per square foot per hour). A comparison of the median reported grate loading factors of plants using the three types of grates and handling the same type refuse (garbage and combustible rubbish) ranged from 64-76 pounds per square foot per hour, a variation of less than 20%.

Furnace Combustion and Flue Chamber Design Factors

The variation in furnace volume and secondary chamber design factors, while substantial, was not so pronounced in the various types of furnaces as in grate design factors. Listed in Table IV are the furnace volumes in terms of cubic feet per ton of rated capacity; and the ratio of the combustion chamber volume and the flue volume respectively to the furnace volume according to type of grate and type of refuse burned.

For furnaces utilizing rotary circular type grates the median values of furnace volume design factors reported ranged from 9.6 to 11.6 cu. ft./ton of rated capacity for garbage and combustible rubbish, mixed refuse, and combustible rubbish. The corresponding ratios of the combustion chambers and the flues to the furnace volume in the "median" plants were as follows:

Figure 3 GRATE TYPE USED VS. TIME (3YR. GROUPS) OF COMPLETION OF PLANT CONSTRUCTION (FOR 92 CITIES)

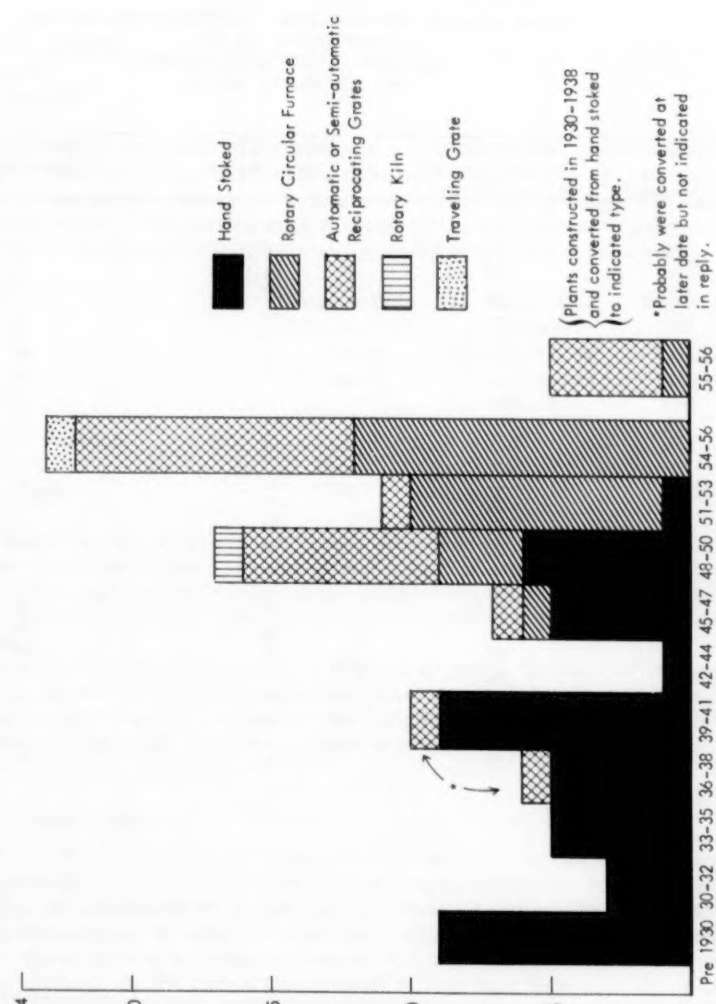


TABLE III

GRATE LOADING DESIGN FACTORS REPORTEDLY USED IN THE
CONSTRUCTION OF 41
MUNICIPAL INCINERATOR PLANTS*
lbs. per sq.ft. per hr.

Type of Refuse Burned	Automatic or Semi-Automatic Sloped Grates	Rotary Circular Type Grates	Hand Stoked Type Grate
GCR	83	120	111
GCR	71	100	100
GCR	57	84	70
GCR	--	83	70
GCR	--	70	65
GCR	--	50	64
GCR	--	42	56
GCR	--	25	50
GCR	--	--	46
GCR	--	--	40
GCR	--	--	15
CR	--	100	--
CR	78	100	--
CR	51	80	--
CR	44	72	--
R	71	150	70
R	63	84	60
R	60	83	47
R	50	--	--
R	48	--	--
G	58	--	--

*One city reported a grate loading of 70 lbs per sq.ft. per hr. used in a rotary kiln type plant burning garbage and combustible rubbish.

CODE: GCR - Garbage and Combustible Rubbish
CR - Combustible Rubbish
R - Refuse
G - Garbage

	Furnace Vol. Cu. Ft./Ton of Rated Capacity	Ratio of Combustion Chamber Vol. to Furnace Volume	Ratio of Flue Vol. to Furnace Volume
G & C Rubbish	9.6	2.8	---
Refuse	10.0	0.6	0.6
C Rubbish	11.6	1.3	6.9

For furnaces using automatic or semi-automatic sloped grates the median values of furnace volume design factors ranged from 13.6 to 15.0 cu. ft./ton of rated capacity for garbage and combustible rubbish, mixed refuse, and combustible rubbish. The corresponding ratios of the combustion chambers and the flues to the furnace volume in the "median" plants were as follows:

	Furnace Vol. Cu. Ft./Ton of Rated Capacity	Ratio of Combustion Chamber Vol. to Furnace Volume	Ratio of Flue Vol. to Furnace Volume
G & C Rubbish	13.6	1.2	0.05
Refuse	15.0	0.8	0.13
C Rubbish	15.0	1.9	1.9

It is interesting to note that although the reported furnace volume design factors for the hand stoked plants varied from 5 to over 23 cu. ft. per ton of rated plant capacity, the median of the plants designed to incinerate garbage and combustible rubbish had a design factor of 12.0 cu. ft. per ton of rated capacity.

Based on the data reported, it is difficult to select "preferred" design factors. In fact, the data is consistent, generally, only in its inconsistency. It would seem a logical conclusion that further field investigation and research is needed to establish desirable standards of basic design for furnace, combustion, and flue chambers.

Stack Design Ratios

Thirty-six cities reported data on their newest or best plant which enabled the computation of the stack design volume and comparison with the rated furnace capacity assigned to each stack. Table VI shows this comparison or ratio according to the type of refuse the plants were reportedly designed to burn. Again there was a wide variance. Some of this variance can be explained perhaps by the use of forced draft rather than natural draft. Although the data reported did not specify natural draft, design for day to day operation interpretation of that which was reported indicated that the vast majority relied on natural draft for day to day operation. It should be recognized further, however, that extremes computed in this data may well be the result of inaccurately reported data due to misinterpretation of the questionnaire or other factors. A summary of the stack design ratio data is presented in Table V.

TABLE IV

FURNACE COMBUSTION AND FLUE CHAMBER DESIGN FACTORS
REPORTEDLY USED IN THE CONSTRUCTION OF 36 MUNICIPAL INCINERATORS

Type of Refuse Burned	AUTOMATIC OR SEMI-AUTOMATIC SLOPED GRATES			ROTARY CIRCULAR TYPE GRATES			HAND STOKED TYPE GRATE		
	Furnace Vol. Cu. Ft./Ton of Rated Capacity	Ratio Combustion Vol. to Furnace Volume	Ratio Flue Volume To Fur. Volume	Furnace Vol.Cu. Ft./Ton of Rated Capacity	Ratio Combustion Vol. to Furnace Volume	Ratio Flue Volume To Fur. Volume	Furnace Vol. Cu. Ft./Ton of Rated Capacity	Ratio Combustion Vol. to Furnace Volume	Ratio Flue Volume To Fur. Volume
GCR	24.0	1.3	3.16	11.0	1.4	1.1	23.2	0.40	0.40
GCR	13.6	1.2	0.05	10.4	1.9	—	15.0	0.67	—
GCR	10.0	0.96	—	10.0	0.60	0.6	12.2	1.4	0.07
GCR	—	—	—	10.0	0.45	0.9	12.0	1.0	—
GCR	—	—	—	9.9	2.9	—	11.0	1.5	1.5
GCR	—	—	—	9.2	2.8	—	6.8	—	—
GCR	—	—	—	9.2	1.7	0.81	5.0	—	—
GCR	—	—	—	9.0	1.9	1.0	—	—	—
GCR	—	—	—	8.0	2.5	—	—	—	—
GCR	—	—	—	7.0	2.1	—	—	—	—
CR	21.6	2.1	1.6	13.4	2.0	3.5	—	—	—
CR	15.0	1.9	1.9	11.6	1.3	6.9	—	—	—
CR	14.0	2.3	14.8	7.5	1.8	2.0	—	—	—
R	17.0	2.4	0.10	11.0	2.9	2.3	16	1.1	0.74
R	15.0	0.8	0.13	10.0	0.6	0.6	13	1.5	0.31
R	14.0	2.1	—	9.0	1.9	1.0	—	—	—
R	10.0	1.5	4.85	—	—	—	—	—	—
G	5.8	0.25	0.43	—	—	—	—	—	—

CODE; GCR - Garbage and Combustible Rubbish
CR - Combustible Rubbish
R - Refuse
G - Garbage

Table V

Summary of Stack Design Ratios
According to Type of Refuse Burned

	Range of Ratios (Stack Volume to Rated Cap.)	Median Plant	No. of Cities Reporting
Garbage and C. Rubbish	3.6 - 311	29	20
C. Rubbish	11.7 - 90.7	55	6
Mixed Refuse	9.4 - 181	51	9
Garbage	2.6	--	--

Charging Methods and Plant Storage Capacity

Satisfactory combustion depends not only on proper design but also, in large part, on proper operation. In recent years, the need for plant layout to facilitate storage of refuse as delivered to the plant in order to permit uniform charging over the operating day has been demonstrated to be of prime importance. With a few exceptions (depending primarily on climate and plant location with a city) pit and crane design is utilized to achieve sufficient storage capacity.

Fig. 4 is a graphical presentation of the charging methods used in 100 municipal incinerators according to population group. As might be expected the vast majority of the plants constructed in cities having more than 100,000 population are of the pit and crane design. Recently, conveyor systems to further facilitate uniform charging of the furnaces have been introduced. It is interesting to note that a substantial number of crane and bin type plants have been built in recent years in less than 25,000 population group.

Listed in Table VII are data on the storage pit capacity, reported in 39 plants, as compared with 24 hour rated capacity. The data is broken down according to type of refuse burned in the plant and the year construction was completed. Again, the most constant factor seems to be the varying ratios with no particular relationship to whether the plant is many years old or almost new. The latter would seem to be particularly pertinent in view of the sharply increased volume of rubbish being produced in urban life in recent years. Data on 19 plants incinerating garbage and combustible rubbish showed a variance of pit storage capacity from 3-1/2 hours to 1-2/3 days, the median plant reporting a ratio of pit volume to rated plant capacity of 3.3 (16 hours assuming the density of GCR as 350#/cu. yd.). Fifteen plants reportedly burning mixed refuse reported pit storage capacities indicating ratios of 1.3 to 28.0. The median plant reported a ratio of pit volume to plant rated capacity of 4.0 which represented a 24 hour storage capacity assuming the density of mixed refuse as 400#/cu. yd. A comparable amount of data on combustible rubbish and garbage incineration plants was lacking, but that which was reported is included in Table V.

Use of Licensed Engineer Operators

Ninety cities reported on whether licensed engineer operators were required in their municipal refuse incinerator plants. Eighty-five replied negatively. Atlanta, Georgia; Minneapolis, Minnesota; Gloucester City, New Jersey; Trenton, New Jersey; and Providence, Rhode Island, replied affirmatively.

TABLE VI

STACK DESIGN RATIOS
A COMPARISON OF STACK VOLUMES AND RATED CAPACITIES ACCORDING TO TYPE OF REFUSE BURNED

Type Refuse Burned	Rated Capacity Tons/day/stack	Stack Cross Sectional Area (sq.ft.)	Stack Ht.(ft.)	Stack Vol. (Cu.Ft.)	Ratio Stack Vol. to Rat.Cap.
Garbage and	500	78	176	13,700	27.4
Combustible	450	113	175	19,800	44.0
Rubbish	400	28	100	2,800	7.0
	300	79	183	14,500	48.4
	300	50	116	5,800	19.3
	300	100	179	17,900	59.7
	300	86	167	14,400	48.0
	300	76	175	13,300	44.3
	250	9	100	900	3.6
	240	28	100	2,800	11.7
	225	227	126	28,600	126.0
	200	50	125	6,250	31.3
	192	50	128	6,400	33.3
	192	16	80	1,280	6.7
	125	33	90	2,970	23.8
	100	20	100	2,000	20.0
	72	14	70	980	13.6
	60	14	83	1,160	19.3
	30	12	87	1,040	34.7
	27	138	61	8,400	311.0
Combustible	300	50	129	6,450	21.5
Rubbish	300	133	204	27,200	90.7
	250	95	166	15,800	63.2
	150	52	140	7,280	48.5
	120	20	70	1,400	11.7
	100	50	146	7,300	73.0
Combined Refuse	480	36	125	4,500	9.4
	400	150	175	26,200	65.5
	375	452	150	67,800	181.0
	300	113	150	16,900	56.3
	300	95	160	15,200	50.7
	225	50	125	6,250	27.8
	200	45	100	4,500	22.5
	200	75	175	13,100	65.5
	50	9	88	792	15.8
Garbage	150	13	30	390	2.6

Figure 4 CHARGING METHOD USED IN 100 MUNICIPAL INCINERATORS ACCORDING TO POPULATION GROUP

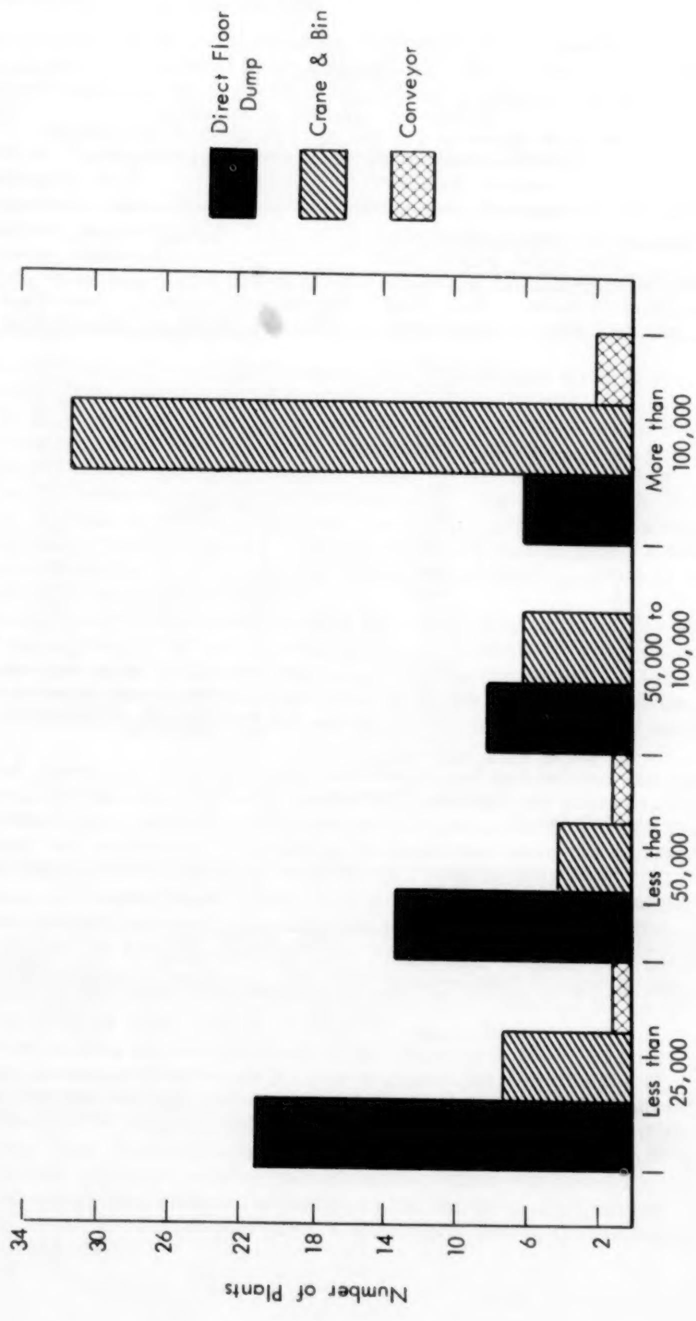


TABLE VII

STORAGE PIT CAPACITY OF 39 PLANTS
RATIO OF VOLUME OF PIT (CU.YDS.) TO RATED PLANT CAPACITY
(TONS/24 HRS.) ACCORDING TO TYPE OF REFUSE BURNED

GARBAGE AND COMBUSTIBLE REFUSE		REFUSE	
Ratio of Pit Vol. (Cu.Yds.) to Rated Cap. of Plant	Year Plant Constructed	Ratio of Pit Vol. (Cu.Yds.) to Rated Cap. of Plant	Year Plant Constructed
8.3	1949	28.0	1954
6.3	1936	10.0	1935
5.8	1955	6.0	1954
5.3	1955	5.9	1956
4.2	1950	5.0	1955
4.1	1954	5.0	1951
4.0	1940	4.7	1956
3.3	1951	4.0*Median	1955
3.3	1939	4.0	1955
3.2*Median	1939	3.9	1929
3.0	1956	3.2	1954
2.5	1954	3.0	1956
2.2	1953	2.9	1948
2.0	1948	1.9	1938
1.7	1941	1.3	1955
1.6	1950		
1.2	1952		
1.0	1939		
0.7	1955		

*The median value represents 16 hour storage capacity assuming the density of GCR as 350#/cu.yd.

*The median value represents 24 hour storage capacity assuming the density of R as 400#/cu.yd.

COMBUSTIBLE RUBBISH		GARBAGE	
Ratio of Pit Vol. (Cu.Yds.) to Rated Cap. of Plant	Year Plant Constructed	Ratio of Pit Vol. (Cu.Yds.) to Rated Cap. of Plant	Year Plant Constructed
6.0	1954	0.3*	1948
3.3	1956*		
1.3	1950		

*Assuming a density of 200#/cu.yd. this value represents about eight hour storage capacity.

*This value represents a 2-1/2 hr. storage capacity assuming the density of G as 1000#/cu.yd.

Operational Employee Training Courses

Seventy-seven of the 89 cities reporting on whether formal operational employee training courses were given indicated no. The twelve cities listed below reported that some type of formal operational employee training courses were given:

Hartford, Connecticut	Youngstown, Ohio
Washington, D. C.	Portland, Oregon
Jacksonville, Florida	Memphis, Tennessee
Brookline, Massachusetts	Fort Worth, Texas
St. Louis, Missouri	Fond du Lac, Wisconsin
Omaha, Nebraska	Shorewood, Wisconsin

Costs, Construction and Operation

Construction costs of municipal refuse incinerators depend not only on general construction cost trends but on many other local factors. For example, a direct floor dump plant can be constructed at a substantially lower first cost than a pit and crane type plant. Similarly, construction in a year-round warm dry climate permits design of a plant without many costly appurtenances, including sometimes the building itself. For example, the Ft. Lauderdale, Florida, incinerator plant was constructed in 1954 at a unit cost of $1,700 per ton of rated capacity. This relatively low unit cost was achieved through limited building construction and appurtenances made possible by the climate and the plant's isolated location.

Fig. 5 shows construction costs (dollars per ton of rated capacity) for 59 municipal incinerator plants built during the period 1940-1956. The relatively sharp increase in costs is self-evident, particularly in the period 1948-1956. In 1948 the median plant cost was in the area of $2,000 per ton of rated capacity. In 1956 this figure was $3,300 and apparently increasing at an increasing rate.

Reported operational costs proved to be quite varied, but fell into a general geographical area pattern. Listed in Table VIII is a summary of personnel and maintenance costs according to geographical location for 60 cities. The normal range was arrived at by computing the upper and lower 1/4 points as well as the median of the reporting cities in each geographical area. The median values of the Northeast, North Central and Southern areas were 252, 261, and 242 dollars per ton of rated capacity per year respectively. The median value for the Pacific Coast area was appreciably higher, 424 dollars per ton of rated capacity per year. The latter value should be interpreted keeping in mind that only 6 cities (five from California) reported the necessary data to compute these costs in the Pacific coast region.

Of further interest in analyzing costs in the effect of 8, 16, or 24 hour operation on plant maintenance and personnel costs. Data supplied by 61 cities permitted plotting costs of operation and plant maintenance as shown in Fig. 6. This data indicates a decided advantage insofar as plant maintenance and personnel costs are concerned for 16 hour operation over 8 hour operation and also 24 hour operation over 16 hour operation. Thus, the data indicates strongly that where it is possible to design a plant for 24 hour utilization rather than 8 hour or 16 hour operation, a substantial saving in operational costs may be realized.

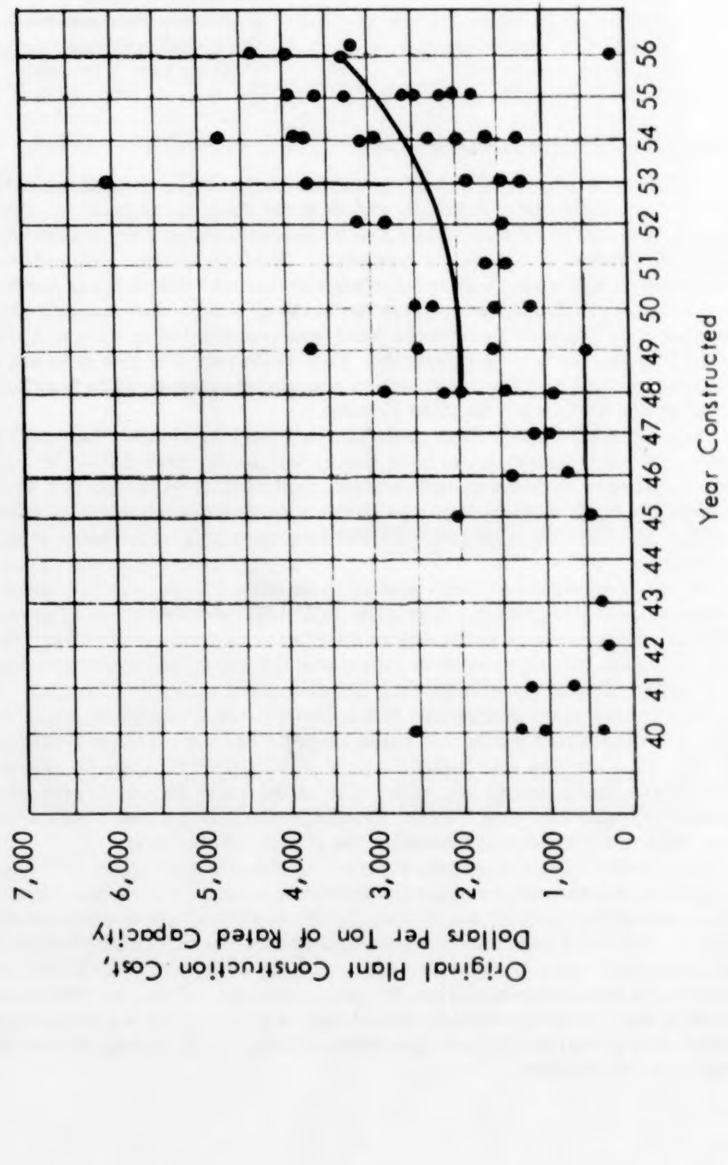


Figure 5 CONSTRUCTION COSTS FOR 59 MUNICIPAL INCINERATORS, 1940-1956

Figure 6 AFFECT OF 1, 2 AND 3 SHIFT OPERATION ON OVERALL PLANT MAINTENANCE AND PERSONNEL COSTS* IN 61 CITIES, 1956 (ON A PER FURNACE BASIS)

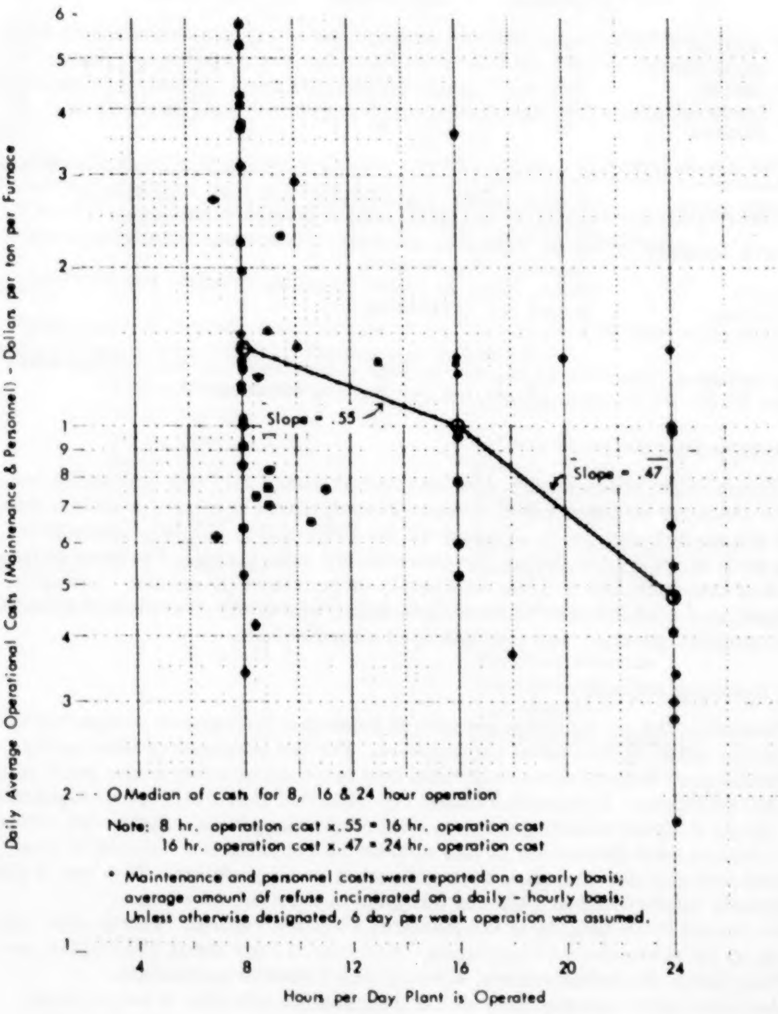


TABLE VIII
A SUMMARY OF PERSONNEL AND MAINTENANCE COSTS
ACCORDING TO GEOGRAPHICAL LOCATION*
(DOLLARS PER TON OF RATED CAPACITY PER YEAR)

	North East (22 Cities)	North Central (21 Cities)	South (11 Cities)	Pacific (6 Cities)
Maximum	580	714	393	716
Upper 1/4 pt.	388	521	350	550
Median	252	261	242	424
Lower 1/4 pt.	218	153	104	143
Minimum	98	90	94	120

*North East:	Massachusetts, Connecticut, New York, New Jersey, Delaware, Maryland, and District of Columbia.
North Central:	Ohio, Illinois, Michigan, Wisconsin, Minnesota, Missouri, and Nebraska
South:	Texas, Virginia, West Virginia, Georgia, and Florida
Pacific:	Oregon and California

The median of reported 8, 16, and 24 hour operational and maintenance costs were \$1.40, \$1.00, and .48 per ton per furnace/day respectively.

Auxiliary Fuel

Eighty-eight cities reported on whether auxiliary fuel was ordinarily needed to complete incineration of refuse. Seventy-three indicated auxiliary fuel was not needed ordinarily, although 13 (or 17%) of this number indicated it was used at least once during the previous full year (1956). Fourteen cities replied that auxiliary fuel was ordinarily or occasionally needed. Ten of the 14 indicated garbage and combustible rubbish was the type of refuse burned; two specified garbage; and two specified mixed refuse.

Air Pollution Aspects

Recently, the air pollution aspects of municipal incinerator design and operation has taken an increased significance. For the purposes of this survey, potential plant nuisances were divided into three categories; noise, odor, and fly ash problems. Eighty-four cities reported that they received no complaints in regard to noise resulting from operation of their plants. Eighty-six cities reported on odor nuisances; of this number 85 reported the absence of complaints and only one indicated odor nuisances was a problem. This was a plant originally constructed in 1939 and designed to burn mixed refuse.

In regard to fly ash, 51 of the potential 87 cities replied. Thirty-nine indicated no fly ash nuisance complaints. However, 12 (or about 25%) of the reporting group did indicate there were fly ash nuisance complaints.

As a matter of general interest the responsible officials in the various cities were queried as to whether air pollution standards were taken into account when the plant was designed. Fifty-five replied that some type of air pollution standards were taken into account. Twenty-two (or about 30%) of the reporting cities indicated such standards were not. Of the 55 cities stating that air pollution standards were taken into account, 43 indicated that their plant complied. Five indicated that their plant did not, and six indicated this

to be unknown. The air pollution standards given as points of reference varied from simple use of a ringleman chart as a base reference to such complex criteria as 0.30 gr./cu. ft. corrected to 50% excess air and 500° F.

CONCLUSIONS

The considerable variety in the construction and operational characteristics of municipal incinerators are indicative of a need for further research and development to assure more efficient design parameters.

A definite trend toward further mechanization has been noted in the survey data.

There is an apparent need for more careful training of technical and other personnel concerned with municipal incineration.

The problem of air pollution caused by the incineration process is achieving considerable recognition and demands additional investigation.

Credit

This research report, which is one of a series of professional contributions by the Committee on Sanitary Engineering Research,

	E. R. Hendrickson)	Air Pollution
	W. T. Ingram)	
	M. A. Churchill	Stream Pollution
	R. W. Bogan	Sewage
	C. H. Hull	Water
	H. A. Faber	Public Health
	R. Stone	Solid Wastes Engineering
Chairman,	N. L. Nemerow	Industrial Wastes

has been prepared by the Solid Wastes Engineering Section

Frank Bowerman
John Merrell, Jr.
Leo Weaver*
Ralph Stone, Head

*Primary responsibility for the compilation and analysis of the data and preparation of the manuscript.

Table IX

Reporting Cities

Alabama
1. Birmingham

California
2. Alhambra
3. Los Angeles
4. Pasadena
5. Santa Monica
6. Signal Hill

Connecticut
7. Darien
8. Greenwich
9. Hartford
10. Middletown
11. New Britain
12. New Canaan
13. Waterbury

Delaware
14. Wilmington

District of Columbia
15. Washington

Florida
16. Coral Gables
17. Ft. Lauderdale
18. Jacksonville
19. Miami

Georgia
20. Atlanta

Illinois
21. Chicago
22. Oak Park

Iowa
23. Clinton

Louisiana
24. New Orleans

Maryland
25. Baltimore
26. Frederick
27. Salisbury

Massachusetts
28. Brookline
29. Fall River
30. Framingham
31. Lawrence

Massachusetts (continued)
32. New Bedford
33. Newton
34. Waltham
35. Worcester

Michigan
36. Detroit
37. Highland Park
38. Muskegon
39. Oakland County

Minnesota
40. Minneapolis
41. Montevideo
42. St. Cloud
43. St. Louis Park

Missouri
44. St. Louis

Nebraska
45. Omaha

New Jersey
46. Gloucester City
47. Princeton
48. Trenton

New York
49. Babylon
50. Buffalo
51. Corning
52. Hempstead
53. Huntington
54. Mechanicville
55. Middletown
56. New York City
57. Rochester
58. Schenectady
59. Tonawanda
60. Yonkers

North Carolina
61. Concord
62. Durham

Ohio
63. Barberton
64. Bedford
65. Cincinnati
66. Cleveland

Ohio (continued)

67. Cuyahoga Falls
68. Dayton
69. East Cleveland
70. South Euclid
71. Warren
72. Youngstown

Oregon

73. Portland

Pennsylvania

74. Beaver Falls
75. Oil City
76. Philadelphia
77. Sewickley

Rhode Island

78. Newport
79. Providence

Tennessee

80. Jackson
81. Memphis

Texas

82. Fort Worth
83. Houston

Virginia

84. Alexandria
85. Arlington
86. Newport News
87. Norfolk
88. Portsmouth
89. Radford
90. Staunton

West Virginia

91. Charleston

Wisconsin

92. Fond Du Lac
93. Green Bay
94. Kenosha
95. Merrill
96. Milwaukee
97. Monroe
98. Oconomowac
99. Sheboygan
100. Shorewood
101. West Bend

102. Winnepeg, Manitoba

AMERICAN SOCIETY OF CIVIL ENGINEERS
SANITARY ENGINEERING DIVISION
Committee on Sanitary Engineering Research

March 1, 1957

Dear Sir:

Recently questions have arisen concerning the adequacy of the basic incinerator plant design criteria available to meet today's refuse disposal requirements. The refuse section of the committee on sanitary engineering research is conducting this survey to enable better evaluation of conditions in the refuse incinerator engineering field.

According to the information available to us your city is operating one or more refuse incineration plants. Your assistance in filling out the enclosed survey form will be a substantial contribution toward making available to others the benefit of your experience.

If you have more than one plant operating in your city, you should report on your newest or best plant. Please list any remarks you feel are pertinent on the reverse side of the survey form.

The data will be compiled and analyzed for possible publication in the ASCE Proceedings and will be used by interested public works officials as well as consulting engineers. A copy of the report will be sent to all cities responding to the survey.

Yours sincerely,

Leo Weaver (Signed)

For the Refuse Section
Frank Bowerman
Leo Weaver
Ralph Stone, Chairman

Enclosure

March 1957

American Society of Civil Engineers
Sanitary Engineering Division
Committee on Sanitary Engineering Research

MUNICIPAL INCINERATOR PLANT SURVEY

1.State________________ 2.City________________

3.Plant Name ________________ 4. Year Built ________

5.a.Capacity__________tons/hr.
b.Average amount actually burned __________tons /hr.

6.Material acceptable for burning
a.Rubbish (comb. or non-comb.) ( )
b.Combined or mixed refuse ( )
c.Combustible rubbish only ( )
d.Garbage (waste food) ( )
e.Garbage and comb. rubbish ( )

7.Who was responsible for engineering the plant?
a.Manufacturer ( )
b.Consulting engineer ( )
c.Municipal engineering staff ( )
d.Are you satisfied with plant's performance? Yes ( ) No ( )

8.a.Do you also dispose refuse in sanitary landfills? Yes ( ) No ( )
b.Incinerator ash is disposed in sanitary landfills? Yes ( ) No ( )

9.Design data
a.Do you weigh refuse to be burned? Yes ( ) No ( )
b. Number of furnaces __________
c.Design grate loading __________lb./sq.ft./hr.
d.Grate type: Rotary kiln ( )
Rotary circular furnace ( )
Automatic or semi-automatic reciprocating sloped grates ( )
Hand stoked ( ) Other ( )
e.Furnace volume__________cu.ft./ton of rated capacity
f.Charging method: Direct floor dump ( ) Crane and bin ( )
Conveyor ( )
g.Secondary chamber vol. __________ cu.ft./ton of rated capacity
h.Flue vol.__________vu.ft./ton of rated capacity

9.i.Stack cross-sectional area __________sq.ft.
j.Stack height __________ft.
k.Storage pit volume __________cu.yds.

10.Operational data
a.Are licensed engineer operators used? Yes ( ) No ( )
b. Do you have any type of formal operational employee training courses? Yes ( ) No( )
c.Original plant Cost $__________
d.Maintenance cost per year $__________
e.Personnel cost per year $__________
f.Number of employees __________
g.Number of hours operated daily__________
h.Charge to private haulers of refuse $__________/ton
i.Is there complaint on plant odor nuisance? Yes ( ) No ( )
Noise? Yes ( ) No ( )
Fly Ash? Yes ( ) No ( )
j.Is auxiliary fuel ordinarily needed? Yes ( ) No ( )
If No, was it used last year? Yes ( ) No ( )

11.Stack discharge
a.Were air pollution standards taken into account in designing the plant? Yes ( ) No ( )
If so, does the plant performance comply? Yes ( ) No ( )
Unknown ( )

12.Total number of municipal incinerator plants in operation __________

13.If available, a copy of the original incinerator design report, annual operating reports, and the air pollution standards your plant is requested to meet would be most helpful.

14.Name of reporting official: 15.Date:

____________________ ____________________

Paper 1678 June, 1958 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SED RESEARCH REPORT NO. 19

On	SEWAGE TREATMENT BY LAGOONS
By	The Sanitary Engineering Research Committee, Sewage Treatment Section
From Research Data of	Collected Sources
Acknowledgement	The Sanitary Engineering Division gratefully recognizes the generosity and professional courtesy of the authors quoted in making their research available to the Society for review, presentation, and comment by the Sewage Treatment Section of the Research Committee.

SYNOPSIS

Lagoons have been found to be an effective and relatively inexpensive means of treating domestic sewage. Economic considerations and space requirements tend to make this method of treatment most attractive to small urban areas. Sufficient data and information are now available to provide a conservative empirical basis for design. Additional research is needed to establish a rigorous means of calculating the most economical lagoon requirements and of predicting performance.

INTRODUCTION

Interest in lagoons as a method of sewage treatment has grown steadily during the past 15 years. A number of research groups and regulatory agencies have reported on laboratory and field scale investigations of sewage lagoons. These studies have generally pursued two lines of attack. One

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1678 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

attempts to explain and to understand the basic mechanisms of the process. The other would describe the overall performance characteristics for a given set of conditions. The objective of both has been to provide a basis for design and to establish a means of predicting performance. Unfortunately there appears to be considerable disagreement at this time between theory and practice.

The exact number of sewage lagoons in use in this country at the present time is not known. Judging from reports in the literature, it would appear that at least 400 communities are employing lagoons in some capacity in their sewage treatment process. Lagoons are being used as the sole means of treatment for an increasing number of cities. Design appears to be based largely on surface area loadings expressed in terms of lbs. of BOD/acre/day, or the number of people served per acre. Depth and detention time requirements have been determined for the most part on arbitrary grounds. Water depths ranging from 3 to 5 feet are most common. Preselection of surface area loading and of depth obviously fixes the detention time.

In the following paragraphs the results of a number of typical research efforts have been summarized. Performance data and recommended design criteria are presented. The primary points of concern are surface area loading, BOD removals, and the reduction of coliform bacteria.

Examples of Reported Research Results

Performance

1. Research of D. H. Caldwell[1]

Stabilization ponds or lagoons were investigated as a means of providing secondary treatment of primary treated sewage from a number of military establishments during World War II. A summary of reported BOD removals is given below in Table I.

Table I

Performance of Lagoons Treating Clarified Sewage

* Loading people/acre	Effluent Avg. 5 day 20°C BOD ppm	Location
200	32	Fallon, Nevada
200	25	Crows Landing, Cal.
200	53	Santa Rosa, Cal.
250	26	Crows Landing, Cal.
300	71	Fallon, Nevada
300	73	Crows Landing, Cal.
350	65	Crows Landing, Cal.
500	41	Hollister, Cal.
500	74	Santa Rosa, Cal.
600	106	Hollister, Cal.
700	70	Santa Rosa, Cal.

* Avg. 5 day, 20°C BOD settled sewage applied = 135 ppm. Depth and pond arrangements not reported.

2. Melbourne, Australia Sewage Lagoons—Report of Parker, Jones and Taylor(2)

Experiments were begun in 1940 treating a portion of the sewage from Melbourne. Authors studied the performance of both aerobic and anaerobic lagoons. Experimental facilities consisted of two anaerobic ponds having a total area of 0.42 acres and two aerobic ponds with a total area of 3.62 acres. Both types of pond were operated at an average water depth of 3 feet. Algae were removed by filtration prior to measuring the BOD of pond effluents. Reported BOD loadings and removals are presented below in Tables II and III.

Table II

*Performance of Aerobic Lagoon with Increasing BOD Load

BOD influent ppm	154	172	235	272	289
BOD filt. eff. ppm	14.6	14.8	11	65	226
BOD loading lbs/acre/day	51.7	66.5	65	105	202
BOD removal lbs/acre/day	47.1	61.6	62.4	82.2	54.1
BOD reduction—%	89.6	92.0	95.4	76.2	31.2

*Treating effluent from anaerobic lagoon. Temperature during study ranged from 53 to 67° F.

Table III

BOD Reductions During Anaerobic Treatment

*Detention Time Days	BOD Removal lbs/acre/day
1	1862
2-1/2	900
5	470
10	240

*Average water depth was 3 feet.

Sludge accumulation in the anaerobic lagoon was held to be beneficial. Odors from the anaerobic cells were reported to be negligible after alkaline fermentation was established. The aerobic lagoons reportedly operated free from nuisance odor conditions up to the higher BOD loading levels.

3. Experiences with Raw Sewage Lagoons in North and South Dakota(3)

A study was made of five operating lagoons during the period of January 1955 to June 1956. All lagoons were subject to ice cover during winter months. Nuisance odor conditions reportedly developed during the spring thaw but quickly subsided. The average water depth ranged between 3 and 5 feet. Reductions in BOD and in coliform MPN values have been summarized in the following table.

Table IV

Reported BOD and Coliform Reductions for
Five Dakota Raw Sewage Lagoons

a. Loadings

Lagoon	Kadota, S. D.	Wall, S. D.	Lemmon, S. D.	Maddock, N. D.	Wishek, N. D.
lbs BOD/acre/day	22.9	7.0	6.8	9.3	13.0
pop served/acre	195	62	96	63	160
b. Seasonal Reductions in BOD					
Winter	96.2	43.6	91.2	76.8	70.0
Spring	88.0	87.2	88.4	88.2	73.8
Summer	87.0	88.5	93.0	97.2	89.0
Fall	86.2	85.4	92.8	98.4	---
c. Range in Coliform MPN Reductions					
	>99.9%	83-99.9%	59-99.9%	95-99%	95.9-99.8%

4. Kearney, Nebraska Experimental Lagoon(4)

Neel and Hopkins have described a study carried out at Kearney, Nebraska employing an experimental 10 acre lagoon. Lagoon loading was expressed as a population equivalent of 127 persons per acre based on a raw sewage BOD of 0.16 lbs/capita/day. The average liquid depth during the study was 4 feet. Relatively good vertical mixing prevailed, reportedly, throughout the 4 foot depth.

Lagoon temperatures varied from a maximum of 34° C in July to a minimum of 1° C in December. Algal growth and lagoon turbidity decreased beginning in September and increased again in April. Twenty-four hour anaerobic conditions rarely occurred until early October. From October to April anaerobic conditions were the rule. The lagoon surface froze in December and odors were eliminated except for brief periods of thaw. Raw sewage entering the pond reportedly contained D.O. only about 4.5 hours/day. Reported BOD reductions are given below in Table V.

Table V

Observed BOD Removals at Kearney, Nebraska

Period	BOD Loading lbs/acre/day	BOD Removal lbs/acre/day	% Reduction
1954			
June 14 - Sept. 14	157	128	82
Sept. 15-22	66.5	59.6	90
Sept. 23-29	183	156.5	85.5
Sept. 30-Oct. 4	63.5	54.7	86
Oct. 5-22	181	134	74
Oct. 25-30	217	173	80
Nov. 5-16	19.1	15.1	79

Table V (Continued)

Period	BOD Loading lb/acre/day	BOD Removal lbs/acre/day	% Reduction
1955			
Jan. 6-Mar. 8	39	12.5	32
Mar. 9-19	38	18	47
Mar. 20-29	38	21	55
Mar. 31-April 3	43.5	26	60
April 4-June 13	46	34	74.5

Coliform MPN reductions averaged 96 per cent during the period of investigation. In 50 per cent of the samples analyzed the coliform reduction was greater than 90 per cent. The authors concluded that during periods favorable to algal growth the lagoon afforded reductions in BOD and in coliform bacteria comparable to secondary treatment.

5. Raw Sewage Lagoons at Long Beach, Washington[5]

The city of Long Beach, Washington recently began treating their raw sewage in a series of lagoons. Design appears to have been based largely upon the recommendations of Parker et. al.[2] Sewage is passed first through an anaerobic cell and then into an aerobic cell. The present population of Long Beach is approximately 800 people. The total area of the anaerobic and of the aerobic cells is 0.46 acres and 2.04 acres respectively. Performance data were obtained by the USPHS during two periods in 1957. Available unpublished data have been summarized below in Table VI.

Table VI

Raw Sewage Lagoon—Long Beach, Wn.—Summary of Field Data Reported by G. M. Hansler[5] *

Date	Raw Sewage		Anaerobic Cells (0.46 acres)		Aerobic Cells (2.04 acres)	
	BOD mg/L	Coliforms MPN/100ml	BOD mg/L	Coliforms MPN/100ml	BOD mg/L	Coliforms MPN/100ml
July 8-10, 1957	268	40.8×10^6	160	8.4×10^6	44	6.7×10^6
Sept. 4-5, 1957	255	75.8×10^6	118	7.7×10^6	28	0.2×10^6

*Values shown are based on an average of the reported analytical results of multiple sampling at selected stations within each cell.

During the July investigation, the D.O. content of the raw sewage averaged 0.8 mg/L, that of the so-called anaerobic cells, 0.1 mg/L, and that of the aerobic cells, 12.5 mg/L. Nuisance odor conditions were not reported. From July 8-10, both the ambient and the raw sewage temperatures averaged 60° F while the lagoon temperatures averaged approximately 68° F.

Recommended Design Criteria

Most investigators have attempted to summarize their findings in the form of design criteria. Generally, recommendations have been made regarding

permissible surface loading rates and optimum water depths. Most authors also discuss the significance of pond size and shape and the importance of avoiding short circuiting, etc. While most practicing sanitary engineers are familiar with these latter concepts, they oftentimes may not be as familiar with the design loading criteria and the performance characteristics of sewage lagoons. A number of typical suggested surface area and depth design requirements have been presented in the following paragraphs.

1. Reported Design Practices in the Missouri Basin States(6)

Hopkins and Neel have made an inventory of approximately 100 sewage lagoons in use in the Missouri Basin up to 1956. Table VII, below, gives a summary of reported surface area loadings.

Table VII

Raw Sewage Lagoon Surface Area Loadings—Missouri Basin

People/Acre	No. Reporting
10-50	21
51-80	29
81-110	27
111-200	11
>200	7

2. Loading of Waste Stabilization Ponds in Texas—Towne & Davis(7)

Towne and Davis have summarized the results of a survey of lagoon treatment practices in Texas.(7) Reported surface loadings varied from a maximum of 2280 lbs. BOD/acre/day to a minimum of 2 lbs. BOD/acre/day. The median reported was 32.5 lbs. BOD/acre/day. Apparently most lagoons in Texas are used to provide secondary or tertiary treatment as shown in the following table.

Table VIII

Stabilization Ponds in Texas—After Towne & Davis(7)

Operation	Total Reported	Yes	No
Pretreatment	182	174	8
Primary	-	134	-
Secondary	-	40	-
Recirculation	25	8	17
Effluent Discharge	175	154	21
Odor Complaints	69	9	60
Fish Present	67	31	36

3. Research at the University of Texas(8)

Gloyna and Hermann(8) have recently reported on a comprehensive study of waste stabilization ponds at the University of Texas. Their research was extended to include operation of 5 outdoor pilot plants. These investigators suggest that lagoon loadings should be based on volume, viz. lbs. BOD/acre-ft./day, rather than surface area, as is common today. They conclude that where photosynthesis is the principal criterion, unit surface loadings would

govern, but where temperature and reaction rates are of major concern, then detention time and depth becomes important. According to their findings, temperature and reaction rate generally govern. The range of useful temperatures was found to lie between 3 and 35° C. They have offered the following formula for calculating lagoon volume based on the van't Hoff-Arrhenius temperature—reaction rate relationship.

$$V = 5.37 \times 10^{-8}\, Nq\, y \left[1.072^{(35-T)} \right]$$

Where V = volume in ac - ft
Nq = flow in gallons/day
y = BOD in mg/L
T = operating temperature in °C

The best depth was reported to lie between 2 and 3.5 feet. According to the authors, a sewage lagoon should be designed to provide the desired degree of treatment during the period of minimum annual operating temperatures.

4. Summary of Additional Typical Design Recommendations

Many investigators, in addition to those referred to above, have offered design recommendations based on their findings. Space does not permit a detailed discussion of all published design recommendations. A number of typical design criteria have been summarized for convenience in Table IX. For a more detailed description of these criteria, the reader is referred to the references cited.

SUMMARY AND CONCLUSIONS

Development of lagoons has proceeded largely along empirical lines. Rigorous methods for calculating realistic loadings have been slow in coming. Available performance data indicate that raw sewage lagoons are capable of giving treatment equivalent to conventional secondary treatment processes.

The theoretical concepts commonly employed to explain what occurs during the purification of sewage in a lagoon are in essence the same classicial principles which have been used to describe the self-purification of receiving waters. It is not the purpose of this report to discuss the theoretical aspects of lagoon performance except as they may bear directly on the interpretation of design criteria. For a more detailed theoretical discussion the reader is referred to the literature cited in this review.

Many investigators have placed special emphasis on the significance of algal photosynthetic activity. This has had considerable influence on the current trend in lagoon design. Surface area and depth are both affected. As Oswald and Gotaas have pointed out,[9] where algal growth is important, depth is theoretically limited by the ability of sunlight to penetrate the lagoon liquor. Theoretical depths calculated on the basis of the Beer-Lambert Law generally tend to be less than 2 feet. It is of interest to note that Towne and Davis[7] have reported on several existing lagoons where oxygen production did not equal demand at depths greater than 24 inches.

Wind induced vertical mixing and hydraulic considerations indicate that depths greater than 2 feet may not only be permissible but desirable. Moreover, control of rooted vegetation may limit minimum depths to 3 feet.[1] Based on reports in the literature, most lagoons are being designed to provide an operating depth in the range of 3 to 5 feet.

Table IX

Recommended Design Criteria for Sewage Stabilization Ponds

Surface Loading people/acre	Surface Loading lbs BOD/acre/day	Detention Time days	Liquid Depth feet	Number of Ponds	Remarks	Reference
$\leqq 400$	$\leqq 45$	$\geqq 25$	$\geqq 3$	2 or more in parallel	Clarified or primary treated sewage only	1
$\leqq 100$	$\leqq 20$	-	3 - 5	2 or more in series	Raw sewage	3
$\leqq 133$	$\leqq 26.7$	-	3 - 5		Following primary treatment	3
-	-	1 to 6	In general < 3		Depth & detention time based on available solar energy & on waste characteristics; see ref. for specific eqns. etc.	9
$\leqq 100$	-	120	3 - 5		Raw sewage	10

Seasonal climatic variations appear to exercise a strong influence upon lagoon performance. Yet as long as aerobic conditions prevail, nuisance odor conditions are not produced and treatment performance remains high. In many raw sewage lagoons, deposition of solids and subsequent anaerobic decomposition may account for a greater proportion of the BOD reduction than aerobic oxidation. Furthermore, it is entirely possible that the oxygen requirements of aquatic life growing as a result of sewage nutrients may exceed that of the sewage itself. Much work apparently remains to be done in establishing rigorous means of calculating oxygen requirements and oxygen supply rates. In spite of these obvious limitations, it appears that raw sewage lagoons loaded at the rate of 20 to 40 lbs. of BOD/acre/day should operate free of nuisance conditions up to the point of freezing and ice cover.

Unfortunately, little information is currently available on actual lagoon costs. Hopkins and Neel[6] have reported an average construction cost for 15 lagoons in South Dakota of $13.06/capita. The average land cost was $3.23/capita. It is hoped that more cost data will be made available in the future.

REFERENCES

1. Caldwell, D. H., "Sewage Oxidation Ponds—Performance, Operation and Design," Sewage Works Journal, 18, 433 (1946).

2. Parker, C. D., Jones, H. L. and Taylor, W. S., "Purification of Sewage in Lagoons," Sewage and Industrial Wastes, 22, 760 (1950).

3. Sewage Stabilization Ponds in the Dakotas, Vol. I, Joint Rpt—USPHS, North and South Dakota State Health Depts., 1957.

4. Neel, J. K. and Hopkins, G. J., "Experimental Lagooning of Raw Sewage," Sewage and Industrial Wastes, 28, 1326 (1956).

5. Unpublished field data reported by G. M. Hansler, USPHS, Pacific Northwest Regional Office, Portland, Oregon. (1957)

6. Hopkins, G. J. and Neel, J. K., "Raw Sewage Lagoons in the Midwest," Proceedings Eleventh Purdue Industrial Wastes Conf., p. 197 (1957).

7. Towne, W. W. and Davis, W. H., "Sewage Treatment by Raw Sewage Stabilization Ponds," ASCE Proceedings, Vol. 83, No. SA4, August 1957.

8. Development of Design Criteria for Waste Stabilization Ponds, Civil Engineering Dept., Univ. of Texas, Final Rpt. to the AEC, March 1, 1957.

9. Oswald, W. J. and Gotaas, H. B., "Photosynthesis in Sewage Treatment," ASCE Proceedings, Vol. 81, Sep. No. 686, May 1955.

10. Van Heuvelen, W. and Svore, J. H., "Sewage Lagoons in North Dakota," Sewage and Industrial Wastes, 26, 771, (1954.)

Credit

This research report, one of a series of professional contributions by the Committee on Sanitary Engineering Research,

	William T. Ingram) E. R. Hendrickson)	Air Pollution
	Ralph Stone	Refuse
	R. H. Bogan	Sewage
	C. H. Hull	Water
	M. A. Churchill	Stream Pollution
	H. A. Faber	Public Health
Chairman,	N. L. Nemerow	Industrial Wastes

has been prepared by the Sewage Treatment Section.

R. H. Bogan, Head

Paper 1679 June, 1958 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SEWAGE EFFLUENT USED FOR INDUSTRIAL WATER[1]

Thomas F. Sullivan[2]
(Proc. Paper 1679)

The Odessa Texas Butadiene Plant will require 3 or more million gallons of water per day to meet the requirements of the cooling tower and the boiler make-up system. The only dependable source of water able to satisfy that demand is the Odessa sewage effluent. The pilot plant work carried on this spring was designed to develop through test, the treating system capable of modifying the chemical and physical composition of the Odessa sewage effluent in such a manner as to eliminate those chemical and physical factors that were objectionable in Cooling Tower and Boiler Feed make-up waters.

The principal source of the Odessa city water is deep wells in the Martin County area. The water is pumped to the city, stored, chlorinated, and delivered to the users. The chemical composition of the fresh water is altered during its passage through the city system to the effluent side of the sewage plant by whatever is added to the water by the users detergents, lye, salt, human wastes, bactericides, and other unknowns. Below, in Table I, is given an analysis of the Martin county well water and a 10 day average analysis of the Odessa sewage effluent.

The actual fluctuations in the chemical composition of the well water are not known; but it is expected to change somewhat depending upon the season, rain fall, and which wells are pumped hardest. However, the composition of the influent water is probably reflected somewhat in the changing composition of the sewage effluent, except for Bicarbonate, Chlorides, Phosphates, and Sodium. These four are believed to be a result of the waste products added to the water during passage through the city system and thus will be expected to fluctuate according to factors influencing their use.

Objectionable Chemical Constituents of the Odessa Sewage Effluent

Even a casual glance at this water will tell you that its chemical composition will require extensive and expensive treatment to reduce or remove those

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1679 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

1. Presented at Meeting of Texas Section, ASCE, October 3-4, 1957, Austin, Tex.
2. Div. Water Engr., El Paso Natural Gas Co., El Paso, Texas.

TABLE I

CHEMICAL ANALYSIS OF WATERS

	Martin County Wells	Odessa Sewage Effluent high	average	low
Ca as $CaCO_3$	113	200	187.4	175
Mg as $CaCO_3$	280	240	215	206
Na as $CaCO_3$	388	654	554	546
Total Cations $CaCO_3$		1069	957	809
BiCarbonate $CaCO_3$	220	460	397.7	354
Carbonate $CaCO_3$	0	0	0	0
Hydroxide $CaCO_3$	0	0	0	0
Sulfate $CaCO_3$	291	210	178.4	156
Chloride $CaCO_3$	261	448	383.4	255
Total Anions $CaCO_3$		1069	975.3	809
Total Hardness $CaCO_3$	393	420	403	390
Phenolphthalein Alk.		0	0	0
Methyl Orange Alk.	220	460	395.7	354
Iron Fe	0.56			
CO_2				
SiO_2	51	55	44.5	32
T. D. S. (evap.)	1038		1172	
PH		8.3	8.1	7.8
PO_4	Ortho	35	30.5	25
	Poly	35	31.0	25

factors that are objectionable. Let us discuss these factors as they pertain to this particular water and why they are objectionable.

Calcium and Magnesium

Calcium and Magnesium hardness are the principal constituents of scale. The object of pre-treatment is to reduce the level of these two ions to the point that no scale will form at the expected operating concentration of the particular system.

Sodium

The average concentration of Sodium as Calcium Carbonate in this sewage effluent is 554 ppm. Sodium is not a scale former. It becomes objectionable at higher concentrations in the cooling tower as a corrosion agent when in the form of Sodium Sulfate or Sodium Chloride, and as a deligninfier of wood when in the form of Sodium Carbonate, Sodium Bicarbonate, Sodium Hydroxide, or Sodium Sulfate. Towers have been damaged by these agents attacking the lignin of the wood to the point that complete replacement of the wooden structure has been necessary. Since the Sodium concentration of this water is already high, the pre-treatment used should not increase the Sodium concentration and if possible should reduce it.

Bicarbonates

The Bicarbonate concentration is an average of 383 ppm as Calcium Carbonate. The Bicarbonate salts of Ca & Mg are relatively soluble but the

Bicarbonate ion has the disadvantage of being unstable, particularly as solution temperature climbs. When the Bicarbonate reverts to carbonate or Hydroxide, the concentration of the resultant Ca or Mg salt may be above its solubility level and result in deposition of the compound as scale. The objection to high bicarbonates in feed waters to cooling towers is the previously discussed attack on the cooling tower wood. The third objection lies in its instability at elevated temperatures especially in the boilers where it will revert to Hydroxide and CO_2. The Hydroxide, as Sodium Hydroxide could result in caustic concentrations which would force excessive blowdown rates in the boilers. The CO_2 is corrosive to steam lines and production of excessive CO_2 could result in outrageous corrosion inhibitor costs.

Sulfates

Sulfates are normally limited to a maximum of 1500 ppm as SO_4 in the cooling tower. A high Sulfate concentration in the cooling tower results in an accelerated deterioration of the wood fiber. Sulfuric Acid is frequently used to reduce the alkalinity of cooling tower make-up water so that a maximum alkalinity of 80-120 ppm is maintained. The effect of this practice, while it does reduce scale forming tendencies by producing Calcium and Magnesium Sulfate and reduces the caustic attack upon the cooling tower wood, is to increase the SO_4 concentration of 178. ppm as SO_4 and as will be pointed out later Sulfates become the first limiting factor.

Chlorides

Chlorides are normally undesirable because they contribute to an accelerated corrosion rate. This water carries an average Chloride concentration of 268 ppm as Cl and fluctuated from a high of 313 ppm to a low of 179 ppm. Chloride concentrations in cooling towers should not exceed 2000 ppm.

Silica

Even a very thin silica scale can greatly reduce thermal conductivities and result in tube failures. These scales are usually extremely hard, adherent, glassy in nature, and difficult to remove. Silica concentrations in cooling towers should not exceed 160 ppm.

Phosphates

Phosphate is not normally found in natural ground waters; and its presence in the sewage water is a direct result of waste, the most prominent of which is probably detergents. The effect of Phosphates in a cooling tower depends entirely on whether or not hardness is present and its concentration. In this case, with an average PO_4 concentration of 44 ppm, it is felt that serious Phosphate scale could result from even a low hardness concentration. Therefore, the PO_4 concentration in the made-up water need be held below 3 ppm. Phosphates in the boiler are not serious if they are not spent; in other words, if the phosphates are not in the Calcium form. However, if the feed water contains some hardness, care must be taken to remove the PO_4 from the feed water in order to prevent precipitation of the Calcium Phosphate in the boiler feed lines.

Detergents

Detergents are, of course, a direct result of human contamination. Detergent concentration in the effluent from the activated Sewage Digestion process is expected to run below 10 ppm.

The detergents are expected to be a nuisance in the cooling tower since the water will foam considerable in the basin and the foam may be blown out on the down wind side of the tower. But, the detergents are not expected to effect the heat transfer rates of the cooling water. The effect of detergents upon the relative corrosiveness of this particular water is not known; however, the detergents are not expected to accelerate corrosion rates to any great extent. The detrimental effects of detergents in the boilers is expected to be principally in an increased tendency of the boiler saline to foam and carry over in the steam. We expect to be able to control foaming by the use of an anti-foam agent. The chart below gives the detergent content of the sewage water after the primary clarifier and after partial digestion by aerobic bacteria. The activated sewage digestion was built to operate at 1 gpm., but difficulty was encountered in maintaining sludge flows at desired velocities. Therefore, the unit was never successfully operated.

Samples were collected before and after treatment by the activated sludge for analysis. Results are given in Table 2.

From the Detergent analysis you will note the wide range of detergent concentrations in the sewage water and that the activated sewage treatment reduced sharply this same concentration. It is felt that a properly operated sewage treatment will give greater reductions than were obtained in this operation, below 10 ppm. The method used for detergent analysis is given in the appendix.

Ammonia and Nitrates

Ammonia attacks copper alloys but does not have a great deal of effect upon ferrous metals. The ammonia concentration of the effluent from the activated sludge digestion system is expected to be under 3 ppm as NH_3, and since no non-ferrous alloys will be used in the steam system, the ammonia is not expected to present too great a problem.

The Nitrate concentration in the effluent from the activated sludge digestion system is expected to be below 3 ppm. Nitrates are not dangerous to industrial systems at low concentrations except as they may revert to NH_3 or Nitric Acid and as food for bacteria in open systems. Some further reduction in both ammonia and Nitrates can be expected in the cold and hot process units.

Various Types of Treaters Tested

After deciding what modifications of the Odessa sewage effluent would be desirable or necessary, tests were conducted with various types of conventional pre-treators on a pilot plant scale to determine the actual performance of each with this particular water.

Cold Line Soda Ash Treatment

The Cold Lime Soda Ash Treatment is a common method of reducing Calcium and Magnesium hardness, Alkalinity, Silica, PO_4, and Total Dis-

solved Solids. Normally, it would be an economical composition similar to the Odessa Sewage effluent.

Jar Tests were used as pilot studies in the Cold Lime Soda Ash method of water treatment. The tests serve the purpose of giving a quick indication of what types of chemicals will give the desired results and at what probable concentration in terms of grains per gallon the chemicals must be used. Selected results are given and discussed in Table 3.

TABLE 2

DETERGENT CONTENT, p.p.m.

Date	Primary Clarifier	Activated Sewage Effluent
4-2	115	33
4-3	135	38
4-4	80	25
4-5	100	
4-8	100	26
4-9	32	17
4-10	61	30
4-11	83	15
4-12	75	50
4-13	100	30
4-14	70	41

TABLE 3

TEST I - RESULTS[1]

Lime in Grains per Gallon	Total Hardness	P-Alkalinity	M-Alkalinity	PO_4	P.H.
20	458	184	405	0	11.0
22	420	154	300	0	11.0
24	394	130	264	0	11.0
26	304	196	300	0	11.0
28	348	160	268	0	11.0
30	322	200	300	0	11.0
32	310	193	295	0	11.0
34	326	210	320	0	11.0
35	310	200	315	0	11.0
38	332	220	340	0	11.0
40	338	240	367	0	11.0
42	384	290	453	0	11.0
44	474	300	496	0	11.0
46	504	303	520	0	11.0

[1] ALL RESULTS ppm. as $CaCO_3$

The results of Test I clearly indicate that this water does not react with lime as would be expected. Lime, when added to a normal water will give a grain of hardness reduction per grain of lime when the hardness is present as Carbonate hardness and is Ca. rather than Mg. Floc was heavy and settled easily. The appearance of the Floc. was not what you would normally expect to see; it was very gelatinous and sticky.

Phosphates came down easily and completely. The lowest hardness obtained was 304 ppm and the lowest alkalinity in that range was 295-320 ppm. If this water were treated with lime alone, the make-up water to the cooling tower would have scale-forming tendencies and after acidizing to control alkalinity the SO_4 concentration would be around 510 ppm as SO_4. The expected maximum concentration allowed in the cooling tower would probably be less than 3. Therefore, treatment with lime alone was considered unsatisfactory.

Jar Test number III, (Table 5) indicates that maximum hardness reduction occurs in the concentration around 40 grains of lime and 22 grains of alkalinity. SiO_2 is reduced in the lower Soda Ash concentration to below 35 ppm.; it is believed that the rising concentration of SiO_2 as the Soda Ash dosage was increased confirms the indication of Test II Results — that Soda Ash interferes with SiO_2 reduction in this water. The lowest hardness obtained, 130 ppm, is considered to be unsatisfactory for use in the cooling tower.

TABLE 4

TEST II - RESULTS[1]

Dosages									
Lime[2]	Soda Ash[2]	Alum Sulfate[2]	T-Hardness	P-Alk	M-Alk	OPO_4	TPO_4	SiO_2	P.H.
14	4	5	572	100	576	0	0	45	9.25
14	4	4	532	152	540	0	1	48	9.25
14	4	3	536	132	552	1	1	51	9.25
14	4	2	536	140	548	0	0	52	9.25
14	4	1	516	192	540	0	1	54	9.25
14	4	0	536	156	552	0	0	56	9.25
14	0	4	560	120	448	1	1	46	9.25
14	1	4	560	120	448	0	0	46	9.25
14	2	4	568	128	508	0	0	46	9.25
14	3	4	532	152	540	0	0	45	9.25
14	4	4	532	158	552	0	0	46	9.25

[1] ALL RESULTS EXCEPT PO_4 AND SiO_2 IN ppm. AS $Ca\ CO_3$
[2] ALL DOSAGES IN GRAINS PER GALLON

It will be noted from the first six samples in Table 4 that the SiO_2 rose as the dosage of Alum was decreased. Phosphates were reduced to below 2 ppm. Hardness decreased as alum decreased and alkalinity remained fairly steady. The next 5 samples show that SiO_2 reduction in this test is the direct result of Alum Concentration. Total Hardness and Total Alkalinity were excessively high.

The use of Magnesium Carbonate in conjunction with Lime and Soda Ash did not have any beneficial effect on reduction of Total Hardness, Total Alkalinity, or SiO_2 as shown in Table 6.

The use of Sodium Hydroxide in conjunction with Alum did give excellent hardness and SiO_2 reduction as shown in Table 7, the tremendous increase increase in P and M Alkalinity and Total Dissolved solids more than offset the desirable reduction of hardness and Silica.

In Test number 6 (Table 8) hardness and SiO_2 reduction was satisfactory, but increases in Alkalinity T. D. S. were unsatisfactory. PO_4 reduction under this type treatment was satisfactory.

TABLE 5

TEST III - RESULTS[1]

Dosages Lime[2]	Soda Ash[2]	Alum[2]	T-H	P-Alk	M-Alk	OPO_4	TPO_4	SiO_2	P.H.
40	0	1	300	174	334	0	0	24	10.0
40	5	1	230	340	448	0	0	27	11/0
40	10	1	172	182	367	0	0	26	11.0
40	12	1	150	200	392	0	1	28	11.0
40	15	1	150	220	350	0	0	29	11.0
40	17	1	130	228	350	0	1	29	11.0
40	20	1	130	240	375	0	0	30	11.0
40	22	1	130	248	412	0	0	31	11.0
40	25	1	130	320	482	0	0	32	11.0
40	27	1	140	292	486	0	0	32	11.0
40	30	1	152	330	530	0	0	33	11.0

1 All results except SiO_2 and PO_4 are ppm. as $CaCO_3$

2 All Dosages in grains per gallon.

TABLE 6

TEST 4 - RESULTS[1]

Dosages[2] Lime	Soda Ash	$MgCo_3$	T-Hardness	P-Alk	M-Alk	OPO_4	TPO_4	SiO_2	PH
20	5	2	446	88	368	0	0	52	9.5
20	5	4	450	92	312	0	0	52	9.5
20	5	6	455	120	376	0	0	51	9.5
20	5	8	440	190	450	0	0	56	9.5
20	5	10	430	187	420	0	0	56	9.5
20	5	12	425	140	425	0	0	56	9.5
20	5	14	425	140	420	0	0	56	9.5
20	5	16	425	140	431	0	0	56	9.5

1 All results in ppm as $CaCo_3$ except PO_4 and SiO_2.

2 Dosages in grains per gallon.

TABLE 7

TEST 5 - RESULTS

Dosage[1] Sodium Hydroxide	Alum	T. Hardness	P-Alk	M-Alk	OPO_4	TPO_4	SiO_2	T.D.S.	PH
20	3	314	242	614	0	1	40	1510	9.3
30	3	216	384	732	0	0	30	1690	9.7
40	3	82	448	796	0	0	20	1805	10.0

2 Dosages in grains per gallon

TABLE 8
TEST 6 - RESULTS

Treated with 20 grains of Sodium Hydroxide
Treated with 25 grains of Magnesite
Treated with 1.5 grains of Alum

Total Hardness	58 ppm
P Alkalinity	304 ppm
M Alkalinity	664 ppm
SiO_2	15 ppm
O PO_4	3.5 ppm
T PO_4	2.5 ppm
PH	9.7
T.D.S.	1847

In test 7 the total alkalinity was consistent for cold lime treatment. Total hardness was likewise consistent. PO_4 reduction was excellent. Floc carry-over existed, but was not excessive considering that the operating rate was 3.5 times design capacity.

Hardness changed but slowly; it is noted that after downtime, hardness and alkalinity were lowered. This indicates that a great part of the slime in the cooling towers at a plant using sewage water is probably a result of this same after reaction, and is another reason against lime or lime soda ash treatment alone.

General Comments Concerning Results of the Jar Tests

1) Phosphate reduction to below 3 ppm occurred in all tests and was still good at only 8 grains of lime per gallon.

2) Total Hardness can be reduced to 130 ppm with 35-40 grains of lime and 20-22 grains of Soda Ash. But, 130 ppm of Total Hardness is considered to be excessively hard for the intended service.

3) At the point where lowest hardness was obtained total Alkalinity was approximately 430 ppm. If this concentration of Alkalinity is neutralized by Acid either H_2SO_4 or HCl, the resultant SO_4 or Cl concentration in the cooling tower make-up water will result in low permissible concentration in the tower 2.26-3.0.

4) Floc characteristics were excellent; the pilot plant cold lime Soda treater was run at 3.5 times design capacity without excessive carry-over.

5) At high chemical dosages considerable after-treatment occured; this could develop into a sludge problem in cooling tower basins, dead spaces in heat exchanger and lines, cutting down flow rates, heat exchanger efficiency, and increasing corrosion control problems.

6) Treatment of this water by Cold Lime Soda Ash methods alone is considered to be unsatisfactory.

TABLE 9
TEST 7 - RESULTS

Lime	TH	P-Alk	M-Alk	TPO_4	PH
8	501	122	501	2	9.25
8	507	120	501	2	9.25
8	499	118	498	2	9.25
8	495	118	498	2	9.25
8	498	116	489	3	9.4
8	509	119	488	2	9.25
8	515	108	487	0	9.5
8	517	104	494	2	9.25
8	519	112	499	3	9.25
8	519	117	495	2	9.25
8	520	105	483	0	9.4
8	519	109	491	4	9.25
8	523	112	493	4	9.25
8	525	100	496	1	9.4
8	541	116	500	2	9.2
8	537	132	508	3	9.4
8	540	131	500	3	9.4
8	480	144	482	2	9.4
8	504	138	484	1	9.25
8	448	136	472	1	9.25
8	482	121	474	1	9.25
8	460	118	484	1	9.0
8	476	120	500	0-1	9.25
8	500	116	440	1	9.0
8	512	108	468	0-1	9.0
8	506	112	470	1	9.0
8	503	107	475	1	9.0
8	496	95	482	1	9.0
8	484	88	480	2	8.8
8	499	100	486	1	9.0
8	440	52	460		8.5
8	442	49	458		8.75
8	463	48	457		8.75
8	470	44	452		8.75
8	476	50	466		8.75
8	475	53	477		9.0
8	483	69	480	2	9.25
8	490	72	490	2	9.25
8	496	77	493	1	9.0
8	498	84	500	1	8.75
8	465	100	470	3	8.75
8	440	24	340	1	
8	440	112	436	1	
8	478	87	444	0	
8	500	60	462	0	
8	488	68	460	1	
8	480	73	452	1	
8	480	72	420	2	
8	480	112	448	1	
8	456	92	420	2	
8	463	90	432	0	
8	476	93	451	1	
8	490	101	463	0	
8	526	112	476	0	
8	500	92	472	0	9.0
8	506	66	484		9.0

Clarifier Plus Sodium Cycle Zeolite Exchange

Since hardness and alkalinity were not reduced to desired levels by use of the Cold Lime Soda Ash method, the use of a combination method was tested.

Results expected of the clarifier with a lime dosage of approximately 8 grains per gallon: (1) Reduction of Phosphates to below 3 ppm (2) Reduction of Turbidity to protect the resins and (3) the ability to reduce SiO_2 to 35 ppm if necessary. (See jar test 7, Table 8).

Results expected of the Sodium cycle zeolite exchanger (1) to reduce Total Hardness to below 10 ppm and (2) to have and to maintain a capacity per cu. ft. of at least 23 Kilo grains of Total Hardness as $CaCO_3$.

The tests on the capacity of the zeolite resin were run for 14 days. Data was collected and evaluated. The type used was styrene polyvinyl resin by Chemical Products and Rohm and Hass. The vessel was a 6'' ID by 50'' height polyvinyl chloride plastic unit. The water was the effluent of Test 7 pumped through a pressure filter and thence through the sodium cycle unit. The results of the studies indicated a maximum sustained capacity of 27 Kilo's per cu. ft. No loss of capacity occurred from physical deposits nor did bacterial growths appear on the resin. Use of exchange resin down stream of a clarifier and adequate filters is considered to be a reasonable risk.

Since the Sodium cycle zeolite exchanger has no effect upon Alkalinity, there still remains the necessity of reducing the Alkalinity in the cooling tower by use of Acid and the previously discussed corrosion problems to combat. The Sodium concentration is increased considerably, due to exchanging Sodium Ions for Ca & Mg.

Clarifier Plus a Combination of Sodium Cycle & Hydrogen Cycle Zeolite Exchange Units

The purpose of the clarifier in this treatment series would be the same as for the clarifier plus the Sodium cycle Zeolite exchange units.

The use of a split stream system is particularly adapted to the treatment of this sewage effluent. Since the system operates on the principal that part of the water passes through the sodium cycle unit for removal of hardness and the other portion passes through the Hydrogen cycle unit for removal of Ca, Mg, Na, and K by the following reaction.

$$\begin{matrix} H_2 & & H_2 \\ & \boxed{Zeol} & \\ H_2 & & H_2 \end{matrix} + \begin{matrix} Ca(HCO_3)_2 \\ Na_2CO_3 \\ KCl \\ MgSO_4 \end{matrix} \longrightarrow \begin{matrix} Na & & Mg \\ & \boxed{Zeol} & \\ K & & Ca \end{matrix} + \begin{matrix} H_2SO_4 \\ HCl \end{matrix} + H_2CO_3 \longrightarrow H_2O + CO_2\uparrow$$

As you can see mineral acidity is developed in direct ratio to the concentration of SO_4 and Cl in the influent water. In order to Neutralize this acidity, enough of the Sodium cycle water is blended back with the effluent of the Hydrogen cycle unit to produce an Alkalinity of 10-16 ppm and a PH of 6.0-6.25 after degasification to drive off the excess CO_2. Regeneration of the Hydrogen unit is by H_2SO_4 or HCl and the waste SO_4 or Cl ions rinsed to waste. Thus, by this system there is no need to add either H_2SO_4 or HCl

to the tower for Alkalinity control, and SO_4 or Cl ion's in the cooling tower make-up water are not increased above their original concentrations. Sodium concentrations are not increased because the Hydrogen stream is practically devoid of sodium ions. Total Dissolved solids are decreased since the hydrogen unit effects a complete removal of the Bicarbonate and Carbonate ion along with its component cation; further, in the blended water the reaction of H_2SO_4 and HCl on Sodium Bicarbonate reduces T. D. S. by releasing CO_2 and forming water:

$$\begin{array}{llllllll} 36.5 & & 84 & 58.5 & & & 18 & & 44 \\ HCl & + & Na\,HCO_3 & Na\,Cl & + & H_2CO_3 & H_2O & + & CO_2 \end{array}$$

$$\begin{array}{llllllll} 98.1 & & 168 & 142 & & & 36 & 88 \\ H_2SO_4 & + & 2NaHCO_3 & Na_2SO_4 & + & 2H_2CO_3 & 2H_2O & 2CO_2 \end{array}$$

The testing procedure was the same as used with the sodium cycle units—water was treated in the clarifier, filtered and passed through the Sodium cycle and Hydrogen cycle units. The purpose of these tests was to determine whether or not normal exchange rates would occur and if capacities per cu. ft. were high enough to justify the risk. After 21 days enough data had accumulated to justify the following design data: That the resin has a capacity of 17 kilo grains of Total Alkalinity as $CaCO_3$ per cu. ft. at a regeneration level of 12 lbs. of HCl per cu. ft.; that no loss of capacity was noted and neither inorganic nor organic deposites occurred and no bacterial activity was observed on the resin.

Blend Studies

During the split stream test runs, samples were collected and blends were made to determine the percentage pass necessary through each unit to give a blended alkalinity of 10-16 ppm. The results of those studies are given in Table 10.

TABLE 10
Alkalinity under Various Blends

SODIUM EFFLUENT		HYDROGEN EFFLUENT	T. ALKALINITY	PH'
55 percent	+	45 percent	0	5.3
60 percent	+	40 percent	10	5.6
56 percent	+	44 percent	1	5.4
55 percent	+	45 percent	0	5.3
60 percent	+	40 percent	22	5.8
55 percent	+	45 percent	0	5.3
60 percent	+	40 percent	20	5.8
60 percent	+	40 percent	30	5.8
55 percent	+	45 percent	10	5.5
60 percent	+	40 percent	53	6.1
55 percent	+	45 percent	16	5.7
55 percent	+	45 percent	12	5.6
55 percent	+	45 percent	13	5.7

The results clearly indicate that a blend water of 55 to 60 percent sodium effluent and 40 to 45 percent hydrogen effluent will produce the desired alkalinity of 10 - 16 ppm as $CaCO_3$. The samples listed on the previous page had

not been degasified, thus the PH is lower than would be expected with commercial equipment.

Table 11 shows the expected analysis of the effluent from each of the treaters tested.

These make-up requirements include a 1-% safety factor. The saving in make-up water when using Case III over either Case I or II amounts to 300 gpm less the water required for rinse (58 gpm) or a net water saving of 242 gpm.

TABLE 11
Expected Analysis

	Cold Lime Soda Ash Case I	Clarifier	Clarifier plus Sodium Cycle Zeolite: Case II Sodium Cycle	Clarifier plus Sodium Cycle Zeolite: Case III Hydrogen Cycle	Clarifier plus Split Stream Blended Water
Total H	130	492	0-10	0-10 ppm	0-10
P Alk.	240	110	110	0	0
M Alk.	430	473	473	0	10-16
Cl as $CaCO_3$	383	383	383	383	383
SO_4 as $CaCO_3$	178	178	178	178	178
SiO_2 as SiO_2	35	44	40	40	40
PO_4 as PO_4	0-3	0-3	0-3	0-3	0-3
T.D.S.	1250	1250	1290-1300	523-50	940-950
Concentrations Allowed	2.9-3.0	--	2.9-3.0	--	4.73
Make up to C.T.	1518 gpm		1518 gpm		1218 gpm

Boiler System

The boiler system will consist of 600 psi boilers and 250 psi boilers. Condensate will be returned to a storage tank and the entire make-up requirements of the 600 psi system will be supplied from this tank. The excess condensate over the 600 psi boiler requirements will go to the 250 psi system. This surplus is expected to be 94,300 lbs. per hr.

The operating conditions of a 250 psi boiler are listed below:

Total Dissolved Solids	3500 ppm max.
Total Hardness	0 ppm max.
P-Alkalinity	250 ppm max.
M-Alkalinity	250 ppm max.
SiO_2	50 ppm max.
PH	10-11.0

The detergents carried in the feed water are expected to cause a foaming problem in the 250 psi boilers and the use of an antifoam agent is anticipated. The exact feed rate of antifoam agent will not be known until after the boilers are in operation.

The pre-treatment has been designed with the desire to have a feed water with less than 700 ppm Total Dissolved Solids; this will permit approximately 5 concentrations to be carried in the boiler and prevent impractical blow-down rates. The pre-treatment must reduce hardness to below 5 ppm, reduce SiO_2 to as low a value as may be necessary to be able to stay below the 50 ppm max. set for the boiler at the actual operating concentration of the boiler.

The pre-treating equipment capable of meeting all of these requirements are (1) an evaporator, which is extremely expensive to operate on a system requiring high make-up, (2) a demineralizer, which is both expensive to buy and expensive to operate, (3) a hot process Lime Soda Ash followed by either hot PO_4 treatment or Hot Sodium cycle Zeolite.

The evaporator and demineralizer were rejected because of the equipment and operating cost if used on this particular system.

The hot process lime Soda Ash followed by filters and Hot Sodium cycle zeolite was chosen for testing.

A hot lime reactor was built in order to test the effect of heat upon the reaction of lime and Soda-Ash with the sewage water. The unit operated at 1.5 to 2.0 gpm. Due to difficulty with the pumps and flashing, the system was operated at 170°F; and it is felt that complete reactions did not occur, in other words, that the commercial unit may give greater reduction of hardness and Alkalinity than was obtained in these tests. The hot lime reactor used was of the down comer type and had a sludge recirculation system. The purpose of sludge recirculation is to permit as intimate a contact between the sludge and new water as possible since Silica is removed by adsorption rather than by absorption.

The results of the pilot plant studies demonstrate again that normal reactions do not occur with this water; heat does increase the ability of lime to react with hardness but leaves quite a bit to be desired. SiO_2 reduction was fairly good and commercial equipment should give desired Silica reductions. Total dissolved solids were not as low as hoped for, and if the treated water as a percentage of total feed water exceeds 53-55% then some outside water either from wells or Hydrogen cycle effluent must be used to provide a blended feedwater, having a T.D.S. of below 700 ppm.

TABLE 12
RESULTS OF HOT LIME STUDIES

TIME	LIME	SODA ASH	ALUM SULFATE	TOTAL HARDNESS	P. ALK.	M. ALK.	pH	SiO_2	TOTAL DISSOLVED SOLIDS
12:55	116	--	13	280	172	378	11.0	--	1210
1:15	116	--	15	238	192	334	11.0	--	--
1:35	116	--	18	200	260	390	11.0	--	1159
1:55	47	--	15	172	380	480	11.0	--	--
3:35	47	--	15	168	480	610	11.0	8.75	--
4:10	18	--	0	272	720	--	11.0	12.25	--
4:30	18	--	0	320	760	--	11.0	--	1808
4:55	18	--	0	260	660	--	11.0	8.75	1306
5:25	15	--	0	224	540	--	11.0	8.25	1158
6:00	15	--	0	160	360	460	11.0	--	1118
6:30	15	--	0	140	288	392	11.0	8.50	1230
7:00	13	--	0	107	224	340	11.0	8.25	1156
7:30	13	--	0	120	160	334	11.0	10.25	1143
8:00	13	--	0	136	140	338	10.0	12.25	1138

The Alkalinity of the effluent is expected to be a maximum of 340 ppm as $CaCO_3$. When blended with the condensate the resultant Alkalinity should be greater than can be tolerated in feedwater to a 250 psi boiler; thus it becomes necessary to remove the excess Alkalinity by acidizing and then degasifying to remove the resultant CO_2 in order to protect the boiler feed lines, the boiler and the steam lines from CO_2 corrosion.

It is important to realize that there are many municipalities whose sewage effluent will meet the needs of factories requiring large quantities of high quality water. However, the water should be subjected to as detailed a study as is economically possible by a competent water consultant before commitments are made.

APPENDIX

Total (Sulfate and Sulfonate) Anion-Active Synthetic Detergent Determination in Water

Reagents

Chloroform, C. P.
Methylene-blue solution - 0.035% by weight in 0.01 N H_2SO_4
Sulfuric Acid - 5 N.

Instrument

Dlett-Summerson Colorimeter, filter #54, 44 mm cell.

Glassware

Separatory funnels, 250 ml.
Erlenmeyer flasks 50 ml.
Volumetric flasks, 50 ml.
Funnels
Pipettes, 1 ml. and 5 ml. volumetric type.

Filter Paper

Folded type (S&S #588 12.5 cm)

Procedure: (Standardization)

Prepare a standard detergent solution (anion-active by adding 1.00 grams of detergent to 1 liter of distilled water. 1 ml. of solution contains 1 mg. of detergent.

Pipet 0.5, 1.0, 1.5, 2.0 mg. of standard detergent solutions into 250 ml. separatory funnels. Add 100 ml. of distilled water to each separatory funnel and also to another separatory funnel for a blank. Add 1 ml. of 5 N H_2SO_4 to all separatory funnels followed by 5 ml. of the methylene-blue solution. Invert and shake all funnels to mix reagents.

Add 10 ml. of chloroform to each sample, invert and shake once a second for 25 seconds. Allow to settle so that the complex will settle to the bottom. Separate the Chloroform complex from the water, collecting it in a 50 ml. erlenmeyer flask.

Repeat above extraction twice more, collecting the complex in the same flask.

Place filter paper in glass funnels and place funnels in 50 ml. volumetric flasks. Pass chloroform complex through the filter paper. Wash filter paper with clear chloroform collecting it in the volumetric flasks. Bring volume in flask up to mark with clear chloroform. Then mix the chloroform complex in the flask.

Between 10 and 20 minutes read the samples on a Klett-Summerson Colorimeter with filter #54, using a 44 mm cell. Plot density units against mg. of detergent on graph paper. A straight line will be formed.

Procedure: (Samples)

Repeat steps under procedure (Standardization) beginning with 100 ml. of samples and 100 ml. of distilled water for a blank. It is necessary to run two standard amounts of detergents with each series of samples to check slope of line.

From mg. of detergent in sample and original volume, p.p.m. can be calculated. (1 mg./liter = 1 p.p.m.).

Notes

Sensitivity is 0.2 p.p.m.

If the sample contains thiocyanates more than 5 p.p.m. as CNS and chlorides less than 300 p.p.m. as Cl, add one ml. of silver sulfate solution (0.4% by weight in distilled water) after the addition of the 1 ml. of 5 N H_2SO_4, than procede with the methylene-blue and etc.

If the sample contains more than 5 p.p.m. CNS and more than 300 p.p.m. Cl, the thiocyanates should first be oxidized. After the 1 ml. of 5 N H_2SO_4 has been added, add 1 ml. of 30% Hydrogen peroxide and let stand for 15 minutes, then procede with the methylene-blue and etc.

REFERENCE

Sewage and Industrial Wastes, Vol. 25, No. 1, January, 1953, Anionic Syndets in Amsterdam Sewage, by Degens and etc.

[illegible]

Paper 1680 June, 1958 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

ORGANIZATION OF METROPOLITAN DISTRICTS[a]

Langdon Pearse[1]
(Proc. Paper 1680)

Historical

The government of Metropolitan areas in the United States is ably discussed by Paul Studenski,(1) including a chapter on Special Metropolitan Authorities, their History, and Nature. He states that the origin of this type of Metropolitan organization in the United States cannot be traced with exactness, although the earliest instances are found in Philadelphia in 1790. Both in New York City and Philadelphia the special metropolitan organizations had ceased to exist by 1870. In the second half of the eighteenth century various districts sprang up in Chicago, Boston, and New Jersey, which with some modifications have survived. In 1869, Chicago created three special metropolitan park authorities, and in 1889, The Sanitary District of Chicago was incorporated. About the same time, the Metropolitan Sewerage Commission was established for Boston. With the growth of population since 1890 and the increased attention to sanitation, and, in particular, the development of clean waterways and pure waterways, considerable activity has resulted in the formation of various Commissions, Boards, Districts, or Authorities, looking towards the provision of facilities for two or more municipalities acting together.

In this type of organization, The Sanitary District of Chicago was one of the first in the field of sewage disposal.

In 1894, the first of the port authorities was established, the Portland (Oregon) Port Commission.

These efforts originated through the legal difficulty of cooperating without some method of joining responsibilities by organization, and with the knowledge that in many cases better development can be had through utilities, such as a water system or sewers, planned to serve a drainage area or district rather than individual towns by political boundaries. This tendency was early recognized in England through the formation of River Boards, such as the Birmingham, Tame and Rea Drainage Board, which controls the removal and disposal of sewage of Birmingham and surrounding territory.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1680 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

a. Revised from Report of Sub-Committee on Sewage, Am. Soc. Municipal Improvements, October 24, 1921, L. Pearse, G. G. Earl, and C. M. Reppert.

1. Late San. Engr., The Metropolitan Sanitary Dist. of Greater Chicago (Died July 20, 1956). Completed by Norval E. Anderson, M. ASCE.

Similar Boards in England have been jointly organized under parliamentary acts for the cleaning of streams to prevent nuisance. In the Eastern United States the tendency was originally toward the formation of Commissions organized and empowered by the State to carry out specific work. Among the earlier Commissions were the Metropolitan Sewerage Board in Massachusetts, serving Boston and the metropolitan area around it, and the Metropolitan Water Board. These two were later consolidated into one Board after the major portion of the work had been accomplished. The Water and Sewerage Board has developed largely into an operating organization. On the other hand, in the Middle West, in Illinois, the formation of sanitary districts started with the enabling act in 1889 authorizing the formation of The Sanitary District of Chicago, a separate municipal corporation with broader powers than those of the Massachusetts Commission, being a separate municipality overlying other municipalities, but with a particular purpose and indefinite life. Further, broader bonding and taxing powers were given than in the East. Other enabling acts have been passed in Illinois and Indiana. In New Jersey, the formation of the Passaic Valley Sewerage Commission and such joint projects as the Plainfield, North Plainfield, and Dunellen Sewerage District, have been added to the list.

In the desire to obtain municipal ownership of water works in the State of Maine, the water district idea was developed largely through the efforts of Mr. Harvey B. Eaton, an attorney, who has summarized the history and enabling acts very completely in a paper entitled "Maine Water Districts and Appraisals," published in the New England Water Works Association (Vol. XIX, 147, 1915).

The Maine water districts resemble The Sanitary District of Chicago, in that they were overlying municipalities and were organized, with bonding and taxing powers, for the specific purpose of serving water. Forty-three such districts are now in operation in Maine. Elsewhere might be mentioned the Greater Winnipeg Water District and the Metropolitan Water Board of London (England). There are also a number of private water companies supplying groups of towns.

In a report upon the Metropolitan Water and Sewerage Systems made to the Essex Border Utilities Commission, Ontario, Canada (1917), Morris Knowles has covered very broadly the Metropolitan district idea, pointing out the need of community cooperation, the methods of securing unity action by (1) annexation, (2) extension of municipal jurisdiction, (3) contracts between municipalities, (4) county administration, (5) private enterprise, (6) metropolitan districts. Various examples are discussed, and the difficulties which occur. The methods in vogue of payment of costs are also discussed, chronologically.

General Considerations

This report is intended to summarize in convenient form most of the existing organizations, bringing the earlier information up to date, so far as practicable.

No attempt is made in this report to enumerate the commissions organized to make engineering reports on metropolitan projects, such as the Metropolitan Sewerage Commission (New York City), Commission on Additional Water Supply of New York (Hering-Burr-Freeman), Charles River Dam Commission (Boston) and others. Their field has been a useful one, in most cases paving

the way for construction and leading to broader acts for the construction organization.

In general, this report endeavors to cover largely the districts formed for sewerage or drainage of metropolitan areas, and not the drainage of farm areas. For the purposes of constructing the sewerage system of large metropolitan areas, and not the drainage of farm areas. For the purposes of constructing the sewerage system of large metropolitan areas, and the disposal of sewerage therefrom, with the increase of population and the need of more complete methods, various schemes for organization have been tried, as outlined herein. These may be summarized according to the degree of authority given, and appear to fall under three general groups:

1. Commissions or Boards formed by City Councils for the specific purpose of building intercepting sewers largely and practically within the confines of one municipality. These hardly come within the definition of the metropolitan districts serving two or more municipalities, but are mentioned because of the organization of such Commissions as the Fitchburg (Mass.) Sewer Commission and the Syracuse (N. Y.) Intercepting Sewerage Board, and others.
2. Boards or Commissions appointed by elective officers of Cities, Counties, or States, allotted resources, either limited by specific act or by action of the municipal or Federal bodies governing the municipalities composing the district. Under this classification might be placed such Boards as the Metropolitan Water and Sewerage Board (Mass.), the Passaic Valley Sewerage Commission, the Commission of the District of Columbia, and others.
3. The formation of a sanitary district which is a complete municipality in itself, with powers of taxation and the right to issue bonds, often dependent upon referendum. Frequently the officials thereof are appointed by County or State officers. Under this classification may come the many smaller sanitary districts in Illinois, such as those organized at Decatur, Bloomington, and the North Shore Sanitary District.
4. Sanitary districts organized as municipalities with complete taxing and bonding powers, with referendum on bonds, the governing officials being elected by direct vote of the people residing within the limits of the district. Under this classification comes The Sanitary District of Chicago.

Most of the districts mentioned and discussed in this report are located within the boundaries of one state. However, there has been found a need of some form of organization which will insure cooperation between the districts lying in two or more states. With the growth of pollution in streams and the difficulties in obtaining suitable water supplies, metropolitan difficulties may occur on stream watersheds, both from the water and sewerage standpoint, involving two or more states, as in the case of the Ohio River, where Illinois, Indiana, Kentucky, New York, Ohio, Pennsylvania, Virginia, and West Virginia have joined in a compact to control water pollution, forming a corporate body known as the Ohio River Water Sanitation Commission. Such relations under the governmental organization of the United States can be organized under compacts between states approved by Congress, covering the creation of river or drainage Boards, with powers extending over several states, to coordinate effort and use common policies in matters relating to use of streams for

sewage disposal and the taking of water supply therefrom. This has been the procedure which gradually developed through the last fifty years.

Recently, the formation of an Authority has been permitted by various states, such as New York, Pennsylvania, and Washington, with broad powers to construct, own, and operate toll tunnels, toll bridges, and toll roads, supported by service charges, solely, without any right to levy taxes. One of the largest and most successful of these is The Port of New York Authority, an interstate activity created by New York and New Jersey under a compact dated April 30, 1921. This remarkably Authority reported in 1956 gross operating revenues of $76,712,000.

To finance the necessary works, money must be raised. This is usually done by issuing bonds or by special assessment. Occasionally a share in the general taxes is also designated. Operating and maintenance charges, as well as the payment of interest and sinking fund to retire bonds, when issued, are generally met by taxation or by special assessments, or, in recent years, by charges based upon service.

Originally the special District or Authority was established to furnish facilities for sewerage or sewage treatment or water supply. However, a recent survey shows a wide variety of purposes, including cemeteries, hospital, market, sewers, garbage disposal, fire, power, gas, park, recreation, and war memorial.

Where the right to issue general bonds is given, the amount to be issued is usually fixed, sometimes by definite figures (Fitchburg, Syracuse), but generally in terms of per cent of the valuation of assessed property. In many states this is limited to 5 per cent, an amount fixed at 0.8 of 1 per cent for Indianapolis, and 2 per cent in the general act (1913). In Illinois, The Sanitary District of Chicago has a 3 per cent limit, whereas under the other Illinois acts authorizing the establishment of sanitary districts, a 5 per cent limit is set.

The maximum rate of interest may also be indicated, such as 5 per cent (Indiana), although at Indianapolis a 4-1/2 per cent limit was fixed. The retirement of bonds was also provided for, 20 year installments being common some years ago (Illinois, Indiana). In the Essex Border Utilities Act the limit was set at 30 years, whereas at Indianapolis 50 years was allowed.

Because of the difficulties produced by low and insufficient tax collections during the recent depression, The Sanitary District of Chicago adopted the procedure of selling bonds with a term of twenty years, but retirable at the option of the Sanitary District, as the cash became available.

In the older districts, bonds may be issued without referendum for the corporate purposes. In the districts formed more recently, many acts require a referendum on the bonds, as a whole. Where the act specifically limits the amount in dollars, as a rule the act is the result of public agitation demanding particular relief.

In the case of the Illinois and Indiana sanitary districts, special assessment has not been used to meet construction costs. In the Passaic Valley, however, the cost of construction was assessed or apportioned among the various constituent municipalities roughly in proportion to the service rendered. In the Vancouver sewerage act, 30 per cent of the construction cost is to be assessed upon the municipalities in proportion to their land valuations, and the remaining 70 per cent in proportion to the valuation upon lands benefited. But in determining the division of operating costs, assessment is frequently made against the individual municipalities in the district, which, in turn, is

distributed by taxes or service charges (Boston Metropolitan District, Passaic Valley, Miami Conservancy). In Illinois and Indiana, tax levies are still made through the usual machinery to raise the monies to pay interest, operating and maintenance expense, and provide the funds to retire bonds.

The maximum rate of taxation is usually fixed. There would, however, seem to be a legitimate need for a minimum rate to provide necessary funds for running expense.

In Indiana, for Indianapolis, a levy of two cents on the $100.000 was allowed, plus a levy for sinking fund and interest. The general Indiana Act provides for 0.25 of 1 per cent.

In Illinois, The Sanitary District of Chicago has a taxing power for corporate purposes of two-thirds of 1 per cent of the assessed valuation. Owing to the scaling of the amount by the County Clerk, this was amended to a minimum rate of one-half mill on the dollar. As this proved inadequate in a period of rising costs, the Illinois General Assembly has approved at each session since 1930, a pegged levy established to meet the corporate requirements for maintenance and operation.

The general Illinois sanitary district acts provide in one case for a rate not to exceed one-third of 1 per cent, and in the other, for one-half of 1 per cent, although this latter may be doubled by a referendum vote.

In the Miami Conservancy District, a levy of 0.3 mill on the dollar was provided for preliminary expenses.

The size of the governing board varies widely, from a minimum of three (Illinois general, 1917) to a maximum of thirteen (New Orleans) in the United States, and seventeen (Essex Border) in Canada. Some acts provide that the trustees or commissioners serve without pay. Others pay as high as $7,500.00 per year.

The limits set on the powers of the board vary. Some acts are very explicit on the terms of advertising contracts, etc. Others are not. Ten (10) days is a common limit. Some prescribe thirty-one (31) days. The limit of expenditure without advertising is occasionally fixed. Under conditions prior to 1930, $500.00 was a common figure. At present The Metropolitan Sanitary District of Greater Chicago has a limit of $2,500.00.

In general, the acts appear to have been effective in providing the ways and means for necessary work. Occasionally, where a referendum is required, the project has failed by lack of votes to authorize bonds (Bloomington and Normal, Illinois), or has been delayed by failure to seek authorization of bonds. The limits set by per cents of valuation in some cases have proved inadequate, and in the future should be selected with greater care, to provide necessary funds, both by taxation and by bond issue.

Revenue bonds to cover water or sewage works have become popular as an escape from taxes on real and personal property. During the depression, water revenue bonds in the more prosperous municipalities obtained an excellent rating, inasmuch as the property owners paid for the water used, even though they were inclined to omit paying taxes.

In a situation like that of The Port of New York Authority, with a consistently growing income from utilities with a stable cash payment for service, a high rating results.

Greater Vancouver Sewerage District

The Greater Vancouver Sewerage District was organized in 1913 for the construction and maintenance of the necessary intercepting sewers and sewage

disposal plants for taking care of the urban district around the City of Vancouver.

New Orleans Water and Sewerage Board

The New Orleans Water and Sewerage Board was organized under an Act of the Legislature in 1889. The Act was made operative by vote of the registered taxpayers of the City of New Orleans. The Board is composed of thirteen members, including ex-officio the Mayor, Commissioners of Public Finance, Public Utilities, Public Property, President of the Board of Liquidation, Members of the Board of Liquidation, and seven members representing the seven municipal districts, the latter appointed by the Mayor. These seven members hold office twelve years, serving without pay. There are five standing committees, known as the Executive, Finance, Sewerage, Water and Drainage. The organization is operated under a General Superintendent, who is also Chief Engineer, with six Departments. The clerical work and accounting is placed under the Secretary of the Board. Up to 1953, $89,832,916 had been expended. The expenditures are distributed as follows:

Water Works	$35,015,141
Sewerage	21,931,455
Drainage	32,886,320
	$89,832,916

The Sewerage and Water Board in 1953 is limited to a bonding capacity of 10% of the assessed valuation, which was practically exhausted by the sale of bonds on December 20, 1951, which realized $5,000,250. Its 1953 revenue for construction purposes was derived from a two mill tax and an allocation from a three mill tax, aggregating at present a total of about $3,035,000 per annum. Water rates can be used for the maintenance and operation of the water and sewerage system and to the creation of a reserve fund for renewals and replacements in said systems. Any surplus can be used for the maintenance and operation of the drainage system.

The receipts from water rates in 1952-53 amounted to about $3,300,000. The cost of maintenance and operation of the water and sewer systems is approximately $2,770,000. The contribution to the renewal fund is $480,000. This totals $3,250,000 per year. In addition, there is the cost of maintenance and operation of the drainage system, approximately $650,000, to which the City is expected to contribute.

Syracuse Intercepting Sewer Board

The Syracuse Intercepting Sewer Board was organized under an act providing for the appointment of three Intercepting Sewer Commissioners by the Mayor, and hold office for five years. The Enabling Act authorized and empowered the Board to construct certain intercepting and storm water sewers, and also regulate and improve particular water courses throughout the city. From time to time the Act has been amended to provide for sewage disposal works, additional storm water sewers, and flood prevention. As amended, the Act provides that bonds may be issued up to the amount of $3,200,000, to be sold by the City upon resolution passed by the Board requesting funds in definite amount.

Fitchburg Sewage Disposal Commission

Under Chapter 461 of the Acts of 1910, State of Massachusetts, the Fitchburg Sewage Disposal Commission was organized by the appointment of three Commissioners by the Mayor, with power to construct a system of sewage disposal. Under Chapter 354 of the Acts of 1901, the amount of bonds issuable was limited to $500,000.00, but this limit was extended by a subsequent Act providing for an additional $500,000. The City Engineer was ex officio, the Chief Engineer of this Commission.

Metropolitan District Commission

Under Chapter 350 of the General Acts of 1919, Massachusetts, provision was made for the formation of the Metropolitan District Commission, whereby the Metropolitan Park Commission, existing under Chapter 407 of the Acts of 1893, and the Metropolitan Water and Sewerage Board, acting under Chapter 168 of the Acts of 1901, were consolidated. The Metropolitan District Commission consists of one Commissioner and four associate Commissioners appointed by the Governor with the advice and consent of the State Council. The Commissioner is appointed for a term of five years, and the associate Commissioners are appointed for the term of five years, although the initial term was for one each for one, two, three, and four years. The Commissioner is made the executive and administrative head, and receives an annual salary of not to exceed $6,000.00. The associate Commissioners receive not to exceed $1,000,00 annually. The Commission may be organized into divisions as heretofore, with the usual clerical, police, and engineering force. The expense of maintenance of the Metropolitan Parks, Boulevard, Water, and Sewerage Systems are to be paid for as heretofore, provided, however, that the salaries of the Commissioners and general office expense not clearly chargeable to maintenance work in any one of the districts shall be divided equally between the four.

The original Metropolitan Sewerage Commission was formed in 1889 and the Metropolitan Water Board in 1895.

The total area served by the Metropolitan Water District is 209.3 sq. mi. and includes twenty-five cities and towns, of which twenty-two receive their entire water supply from the Metropolitan system. The population of the towns fully served is 1,621,850, plus an indeterminate portion of the total population of 225,710 of the three cities and towns partially served.

The total area served by the Metropolitan Sewerage District is 302.78 sq. mi. and includes thirty cities and towns, plus a portion of Boston. The population served is 1,527,431. There are 156.757 miles of metropolitan sewers, which receive the discharge from 2,500.41 miles of town and city sewers. The total flow of sewage averaged 211,297 m.g.d. in the year ending June 30, 1951.

Washington Suburban District

Provision was made by Chapter 122 Acts of 1918, Maryland, for the organization of a Sanitary Commission in the Counties of Montgomery and Prince George for the construction of sewerage works. The administration provides for three members, one to be appointed by the Governor, and one by each Board of County Commissioners. Funds are derived by joint taxes for trunk lines and by special assessment for laterals. Service rates are also charged.

Indianapolis Sewerage District

The Act under which the Sanitary District of Indianapolis was organized is Chapter 157 of the Acts of 1917 of the State of Indiana. This Act was written for cities of the first class and creates, in addition to existing executive departments, the Department of Public Sanitation, under the control of a Board of three Sanitary Commissioners, two to be appointed by the Mayor and the third to be the City Engineer ex-officio. One of the two appointees should be an engineer, nominated by the State Board of Health nominate the third man for the approval of the Mayor. The Commissioners serve for four years, each at an annual salary of $3,600.00 during construction and $600.00 after construction. The funds are handled by the City Comptroller and the City Treasurer, and are to be raised by a special tax confirmed by a majority or all of the Superior Judges.

Bonds may be issued from two to fifty years in life, with an interest rate not over 4-1/2% in amount, not to exceed 0.8 of 1% of valuation. The powers given to the Commission are broad and cover and control the construction of sewage disposal works, intercepting sewers, maintenance of operation, and also the handling of garbage, from the standpoint of disposal, but not collection. The Board has the power of condemnation within five miles of the City and can enter for surveying. The Act provides in detail for the legal machinery. The provisions as to the letting of contracts are unusually explicit, providing for the advertising once a week for three weeks, and then 10 days interval to date of receiving of proposals, which are to be accompanied by a deposit of at least 2-1/2% of the amount of the bid. Contracts may be let at less than the estimates of the Board. The Validity of the letting can be questioned only within fifteen days of the execution of the contract. For general expenses, taxes may be levied to the amount of two cents on each $100.00 valuation. In addition, a levy may be made to pay interest and principal on the bonds. Payments to the contractor on partial estimates are limited to 80%.

Sanitary Districts in Indiana

Chapter 307 of the General Laws of Indiana, 1913, is an Act providing for the incorporation of sanitary districts. The original Act provided that two or more incorporated municipalities in any one country might form a sanitary district upon petition of 500 legal voters, and upon the majority vote passed by each municipality in an election held for the purpose. Five Trustees are to be appointed by the Governor of the State. The Act provided for the construction of works, including sewers, sewage treatment plants, and pumping stations, and for the issuing of bonds in an amount not to exceed 2 per cent of the valuation, together with the levying of taxes to pay the interest and principal. The bonds shall not run more than twenty years. All contracts worth over $500.00 shall be advertised fifteen days. Taxes may be levied in an amount not to exceed one-half of one per cent of the valuation of the taxable property. Biennial reports are to be made to the Governor.

The original Act was amended by Chapter 11 of the General Laws of 1919, providing for the appointment of the five trustees by the judge of the Circuit Court of the County in which the district may be located. Each trustee receives an annual salary of $500.00. The power of the Board further provides not only for handling sewers but also for handling and the disposal of garbage and refuse. The issuance of bonds was limited to two per cent of the valuation and the rate of interest was not to exceed five per cent. The tax levy was reduced to one-fourth of one per cent.

The District of Columbia

The District of Columbia, which is the seat of government of the United States, as defined by the Constitution, comprises a territory of seventy square miles, ceded by the State of Maryland to the United States, and located at the head of tide water on the Potomac River. The population in 1950 was 1,457,607.

Under powers conferred by the Constitution, the Federal Congress exercises exclusive and almost unlimited powers over the population within this territory. After a varied experiment for three-quarters of a century with local self-government, in 1878, Congress took away all such rights and placed the District under a Commission form of government, with a Board of three Commissioners appointed by the President of the United States, each for a term of three years, with no restriction as to reappointment. Two of these must have resided in the District for a period of three years preceding appointment, which is subject to rejection or confirmation by the U. S. Senate; the third must be an officer of the U. S. Army, of the Corps of Engineers not below the rank of Major, detailed by the Secretary of War. The salary is $6,000.00 per annum each, since this Board was designated by Congress as a "Public Utility Commission" and granted the powers usual to such Commissions in the various states.

The Commissioners of the District of Columbia constitute a governing board that is purely administrative. Exclusive of their function as a Public Utility Commission, they have no broad governing powers, and what may appear to have considerable scope are in reality so restricted by the acts or the action of Congress as to possess only the barren powers of recommendation. In this connection it is to be noted that both the Senate and the House of Representatives maintain a standing "Committee on the District of Columbia," and in addition, the Appropriations Committee of the House has a permanent sub-Committee, which in effect is authoritative in all fiscal matters.

The Commissions submit the annual estimates for the District of Columbia to the Secretary of the Treasury, who transmits them to Congress. The Committees of both houses consider, in turn, these estimates and report the amount of each item that in their judgment should be allowed. This report constitutes the District of Columbia appropriation bill, which goes first to the House of Representatives out of the House Committee, and when passed by that body, with such amendments as may be made on the floor, then to the Senate for Committee report and consideration by Senate, thence to Joint Conference Committee of the two houses for adjustment of differences, then back to each House for final passage, thence to the President of the United States for approval of this annual appropriation bill.

Not all the amounts thus appropriated in great detail as to items are expended under the direction of the Commissioners of the District of Columbia, but some may come under the exclusive control of the Department of the Federal Government, such as War, Justice, etc. Sixty per cent of all amounts appropriated must be paid direct by local taxes, which, as collected, go directly into the Treasury of the United States. In this aspect of fiscal matters, the sole duty by law of the Commissioners of the District of the Columbia is to fix the local tax rate to insure payment into the Treasury of sixty per cent of the amount decreed for expenditure by Congress.

As Congress fixes just what streets are to be paved and what school houses built and the amount allowed for each individual item, the expenditure of the

appropriations may be considered purely administrative. The Commissioners have no bonding powers, and no taxing powers except as noted, and no referendum is permitted the population on any subject.

The authority and activities of the Commissioners of the District of Columbia are limited entirely to the District of Columbia and to such parts only as are not assigned by Congress to Department or Bureaus of the Federal Government. For example, all the parks, large and small except one, come under the War Department, including the police force for parks. The one exception is the Zoological Park, which comes under the Smithsonian Institution. The water supply is under the War Department, except only the municipal distribution mains and house meters. Charities and correctional institutions come under the Department of Justice or a special board designated by the President. And, for another example, the care of the insane comes under the Federal Department of the Interior, which controls the local asylum of St. Elizabeth's, to which nearly a million dollars a year of local taxes are devoted.

Michigan

Michigan has been unable to follow the statutes existing in Illinois for constituting sanitary districts because of a provision in the Michigan constitution preventing the use of circuit judges in forming such districts. However, under a 1929 Metropolitan District Act, some four districts have been formed; the Beecher Metropolitan District, for water service; the Landell Metropolitan District, for water and sewage service, near Lansing; Bangor-Monitor Metropolitan District, for water distribution only; and the Linwood Metropolitan District, for water distribution.

New Jersey

"The Joint Meeting of the Inhabitants of the City of Plainfield, the Borough of North Plainfield, and the Borough of Plainfield" is the corporate title of the joint municipality which handles the sewage and treats it at Green Brook Park.

The first step was the passage of a law by the State Legislature in 1910 permitting two or more municipalities to join in a contract for the collection and disposal of sewage and authorizing such joint municipalities to acquire land outside of their own boundaries for sewage disposal sites and defining procedure in claims for damages. The second step was the execution of a contract among the three municipalities providing for allocating costs of construction and operation. The division of the cost of construction was somewhat arbitrarily determined, for the reason that the above-mentioned law states that any municipality having paid not less than 25 per cent of the initial cost shall have a veto power on all matters of expenditure. North Plainfield chose to pay this proportion in order to secure such veto power. The division of costs agreed upon in per cent was North Plainfield 25, Plainfield 68, Dunellen 7. All new construction work and enlargements were paid for in this proportion, but the provision was made that enlargements made necessary by the abnormal demands of any one municipality over its proportionate share shall be paid for by such municipality. Provision was made for arbitration of disputes by appointees from each party, these to select a third party, and in case of failure to make such selection, the courts make the appointment.

Money for construction costs was provided by bond issue of the individual municipalities, the money being provided from time to time in accordance with the Engineers' progress estimates. Construction bills were approved by the Engineer, passed upon by the "Joint Meeting" (the entire Council of each municipality in joint session), then passed upon by each Council independently, the money appropriated by each Council separately and paid into the hands of the Treasurer, who then paid the bills by check signed by himself, the President, and the Secretary of the Joint Meeting.

Money for operating expenses is also provided independently by each municipality, but through taxation. The Superintendent of the Joint Works provides an estimate of cost at the beginning of each year. Upon the basis of this estimate, each municipality provides for its share in the annual tax budget. The proportion of operating cost born by each municipality was to be determined, according to the contract, by the flow of sewage determined by Venturi meters, but actually has been divided in accordance with the number of house connections.

During construction the Treasurer never had any funds in hand. In order to expedite the payment of bills for operation, the Treasurer, at the time of each quarterly meeting, requisitions each municipality for a sum sufficient to make up the balance in the treasury to $5,000.00. Bills were paid by the Treasurer on vouchers signed by the Operating Committee. This Committee consisted of a representative of each municipality and met twice each month. The members of this Committee need not be a Councilmen. The contract calls for the appointment of the Committee by the Chairman of the Joint Meeting, but by mutual arrangement, he is guided by the wishes of the various Councils.

Operation is in the hands of the Superintendent, who reports to the Operating Committee. All ordinary purchases are made on order signed by the Superintendent. Larger items of operating expense are authorized by the Committee, whereas improvements, etc., involving several hundred dollars are referred to the Joint Meeting. After approval on roll call of the Joint Meeting, the Operating Committee assumes the payment of the bills in the usual way.

The paid officers are the Secretary of the Joint Meeting, at $300.00 per year; the Treasurer, at $300.00 per year, Members of the Operating Committee at $5.00 per meeting (if Councilmen they are prohibited by law from accepting this sum); and the Superintendent of Disposal Works, who is Engineer and Chemist.

Buffalo Sewer Authority

In 1935, the New York State Department of Health summarily mandated the City of Buffalo to discontinue forthwith the menacing nuisance of pollution of the Niagara River. However, at this time, the bonded indebtedness of the City was so close to the constitutional limit that funds for adequate correction of the prevailing condition could not be provided.

The Buffalo Sewer Authority, a public benefit corporation, was created by an Act of the New York State Legislature in the spring of 1935. To it was delegated responsibility for providing an effectual means of relieving the Niagara River and other tributary streams from pollution by sewage and wastes. The Authority was authorized to borrow money, issue bonds, and provide for their repayment, fix and collect rates and rentals, and in general assume full responsibility for carrying out the State Health Department's mandate in a thorough and efficient manner.

Under the Act, the Authority is authorized to establish a schedule of rates, rentals, and charges, to be called "sewer rents" to be collected from all real property served by its facilities and to prescribe the manner and time at which such rents are to be paid.

The Authority has adopted a schedule of sewer rents which provides generally for revenue, divided into two principal classes:

1. Sewer Rent based upon water use.
2. Sewer Rent based upon assessed valuation of taxable real estate.

In 1951-52, a review of the revenue problem indicated that approximately 58% was the maximum of the total annual operating revenue requirements which could be considered as a general charge and secured from property owners. The procedure used in billing is described in the 1951-52 report.

Sewer Rents		
On Assessed Valuation of Property		
Within City	$1,011,587.68	
Outside City	5,000.00	$1,016,587.68
On Water Use		
Within City	$ 904,658.40	
Outside City	8,000.00	$ 912,658.40
From Other Sources		
Within City	$ 11,000.00	
Outside City—Sewer Connection Agreements	40,037.92	$ 51,037.92
Interest Income	$ 41,600.00	
TOTAL		$2,021,884.00

The actual revenue collected was $2,077,876.54.

The cost of billing and collecting the sewer rents (City) for the fiscal year was approximately 1.34 per cent of the total net billings. In the previous fiscal year, a cost of 1.48 per cent was reported.

The sewage treatment works were built between 1936-1939, with a design capacity for 750,000 population, and an average sewage flow of 150 m.g.d. In 1950, the works were handling an equivalent population of 948,000 and an average flow of 150 m.g.d.

The works cost $5,479,867 and comprise primary settling tanks with sludge digestion, and sludge filtration and incineration. The works effluent is chlorinated.

The Authority also makes a special charge for treatment of industrial wastes of unduly high concentrations which increase the expense of sewage treatment. These special charges are made for the purpose of reimbursing the Authority only for the cost of chlorine and of chemicals and power used for disposal of solids in excess of normal sewage requirements. In 1951-52, the receipts from this source totaled $12,254.86.

The Port of New York Authority

The Port of New York Authority is an outstanding corporate agency, created April 30, 1921 by treaty between the states of New Jersey and New York, with

the consent of Congress, for the purpose of planning and development of terminal and transportation facilities, and to improve and protect the commerce of the Port District which embraces a territory within a radius of approximately 25 miles of the Statue of Liberty.

The Port Authority is governed by a Board of Commissioners, six from each state, appointed by the governors of New Jersey and New York. They serve without pay for terms of six years. The Authority is self-supporting, being financed by revenues from the use of its facilities.

The Authority's Lincoln and Holland Tunnels and George Washington Bridge spanning the Hudson River, and its Bayonne and Goethals Bridges and Outerbridge Crossing connecting Staten Island and New Jersey, join the bi-state metropolitan area for vehicular traffic.

The Authority's terminal facilities include the Port Authority Building, housing the Union Railroad Freight Terminal, the New York Union Motor Truck Terminal and the Port Authority Bus Terminal in Manhattan; the Brooklyn-Port Authority Piers and the Port Authority Grain Terminal at Gowanus Bay, Brooklyn; LaGuardia Airport, New York International Airport and the Port Authority-West 30th Street Heliport in New York City; Newark Airport, Teterboro Airport, Port Newark, the Newark Union Motor Truck Terminal and the Hoboken-Port Authority Piers in New Jersey.

Charged by statute with the protection of port commerce, the Port Authority appears before such regulatory bodies as the Interstate Commerce Commission, the Civil Aeronautics Board and the Federal Maritime Board in the interest of the welfare of the unified Port Area. It maintains branch offices in downtown New York, Washington, Chicago, Cleveland and Rio de Janiero in the interest of promoting the movement of commerce through the Port of New York.

In 1947, the Port Authority signed a 50-year lease with the city of New York and assumed the responsibility for the development and operation of New York international and LaGuardia Airports. On March 22, 1948, a 50-year lease between the city of Newark, N. J., and the Port Authority became effective, for Port Authority financing, development and operation of Newark Airport and Port Newark. On April 2, 1949, it completed the purchase of Teterboro Airport in Bergen County, N. J., to permit full regional development of major airports adequate to handle future air traffic needs of the New Jersey-New York Port District.

The Port Authority started construction in September 1952 of a third tube to the Lincoln Tunnel. This third tube was put into service on May 25, 1957 and cost approximately $94,000,000. It is expected to increase annual traffic capacity of the Lincoln Tunnel by 50 per cent.

In 1955, a $400,000,000 program of bridge and arterial construction over the next five years was recommended in a Joint Report on Arterial Facilities in the New York-New Jersey Metropolitan Area by the Port of New York Authority and the Triborough Bridge and Tunnel Authority. The recommended program is based on a joint study started by the two government agencies in February 1954. It includes a twelve-lane double deck suspension Narrows Bridge, connecting Fort Hamilton in Brooklyn and Fort Wadsworth in Staten Island; a six-lane lower deck of the George Washington Bridge, and a six-lane single-deck suspension Throgs Neck Bridge connecting Cryders Point in Queens and Fort Schuyler in the Bronx. The Joint Report also recommends the construction of extensive connecting highways beyond the immediate approaches to the proposed bridge projects. Such highways would require

financing from sources such as Federal and State funds since they could not be provided on a self-supporting basis.

Also in 1955, the Port Authority announced plans for a "Terminal City" within a 655-acre central landscaped oval at New York International Airport. The development will comprise an eleven-city-block-long International Arrival Building with two adjacent Airline Wing Buildings and individual airline terminal buildings capable of accommodating 140 aircraft at one time. Construction of the International Arrival Building and the two Airline Wing Buildings will be completed in mid-1957. The construction of the airline unit terminals will begin as soon as design plans are prepared and contracts let.

On September 24, 1952, the Port Authority, the city of Hoboken and the United States Maritime Administration signed a three-way lease, effective October 1, for the Hoboken-Port Authority Piers. Under the 50-year lease agreement, the Port Authority assumed responsibility for operation and maintenance of three existing piers and agreed to construct two new piers of modern design.

The Port Authority on January 28, 1954, leased the Hoboken facility to the American Export Lines, Inc., for fifteen years at an annual rental of $1,100,000. On November 30, 1956 the Port Authority's $18,000,000 Hoboken-Port Authority Piers program was completed. In its four years as operator of the Hoboken-Port Authority Piers, the Authority has constructed two wide general-cargo finger piers of the most modern design; has completely rehabilitated an existing double-deck,cargo-passenger pier; and has completely renovated the facility's headhouse.

On March 1, 1956 the Port Authority assumed responsibility for the operation of two miles of Brooklyn waterfront property, known as the Brooklyn-Port Authority Piers. This property was purchased from the New York Dock Company and extends southward from a point near the Brooklyn Bridge to and including Atlantic Basin. This development, the greatest of its kind ever undertaken in the bi-state harbor, comprises construction over a seven-year period of ten new piers, the rehabilitation of an existing pier, construction of three new warehouses, and improvement of fifty acres of upland area. In 1956, the Port Authority Commissioners approved a general plan for the creation of a new marine terminal facility, the Elizabeth-Port Authority Piers, on Newark Bay along the shore-line of the City of Elizabeth.

The construction or acquisition of public improvements is financed by the Port Authority through the issuance of its own bonds. All surplus revenues from the operation of the Port Authority facilities are paid into a general reserve fund to support its bonds. In 1956, net operating revenues totaled $39,617.00; the funded debt was $324,848,000; and the total invested in facilities was $616,298,000.

Passaic Valley Sewerage Commission

Passaic Valley Sewerage Commission is not a municipal corporation but has existed since 1907 under the laws of the State of New Jersey for dual purposes; first, to act as the agent of two or more municipalities, lying in whole or in part within Passaic Valley Sewerage District, in the construction and operation of system of metropolitan sewerage works for the relief of the polluted condition of the Passaic River; and, second, to act, as an arm of the State, to prevent the pollution of the said river, after a date fixed by the Legislature.

There are five Commissioners, one being appointed each year by the Governor, to serve a term of five years. The salary of each Commissioner, as stipulated by law, is $2,500.00 per year. The Commissioners are organized, with a Chairman and a Clerk.

The Passaic Valley Sewerage Commission has no taxing power. Monies for the construction of sewerage works are made available through definite appropriations authorized from time to time by the contracting municipalities, each municipality paying a proportionate share, based on its net taxable valuation. The cost of operation and maintenance is to be apportioned annually among the contracting municipalities, in direct ratio with their respective sewage flows.

The Commissioners act principally under the following laws; as subsequently amended:

District Defined:	Chapter 48, Laws 1902
Enabling Act:	Chapter 10, Laws 1907

Each municipality provides money on its own initiative to pay installments due under contracts with the Commissioners, and may, if it elects, incur indebtedness and issue bonds to an amount not exceeding 5 per centum of its net taxable valuation.

A total of twenty-one municipalities have to date participated in the Passaic Valley Sewerage project.

Miami Conservancy District

The Miami Conservancy District was organized in 1914-1915 under the Conservancy Law of Ohio (104 Ohio Laws, pp. 13 to 64 inclusive, passed February 5, 1914), which was prepared to provide the necessary machinery for works to prevent flood, to protect cities, villages, farms, and highways from inundation, and to authorize the organization of drainage and conservation districts. Briefly, a Conservancy District can be established by petition of the property owners to the Court of Common Pleas in any County. The Court appoints a Board of three directors and also three appraisers. The Miami District includes parts of ten counties. The appraisers appointed by the Court have no executive functions. Their duty is to determine benefits and damages. The Board of Trustees is the governing body of the municipal corporation, with discretion to organize the administrative offices as they deem best. During its active years, the organization of the Miami District was as follows:

> The Secretary of the District made the immediate routine and administrative contact with the Board of Directors and had general supervision of the offices. Under the Secretary were the three Departments. The Department of Taxation was occupied with the clerical work of spreading the assessment on the tax books. The Farm Department was engaged in disposing of the surplus lands which were acquired in the right of way condemnations. Otherwise the entire work of the District was in the hands of the Department of Engineering and Construction.
>
> The Chief Engineer was in direct charge of engineering and construction for the District. He employed men, made contracts (which in important cases were presented to the Board), and was in every respect responsible for engineering and construction.

All bills were finally paid by checks signed by the Directors. Although this was not necessary, this method was adopted to keep in touch with the work. Frequently these vouchers covered large expense accounts, pay rolls, etc., so that the number of vouchers signed was comparatively few.

The Department of Engineering and Construction was separated into two divisions. The Engineering Division exercised the usual functions of the engineer, while the Construction Division exercised the usual functions of the contractor. These divisions were as though the job were let on contract and the responsibility was distributed on that basis. However, there was very close cooperation between the Engineering and Construction Departments, since both were under the direction of the Chief Engineer.

The Bids received for constructing the work were so high that the Conservancy Board on December 3, 1917 decided not to let contracts, and proceeded to build the works with its own forces.

The financing was provided through three funds: (1) A preliminary fund consisting of a tax levy upon the property of the District not to exceed 0.3 of a mill on its assessed value; (2) A bond fund provided by the special assessments of benefits as approved by the Court; (3) A maintenance fund derived from special assessments levied annually.

The first meeting of the Board of Directors was held on July 17, 1915. The construction was completed on April 17, 1923, at a total cost (after sales of land, etc.) of $30,850,000. Thereafter, maintenance has been the only activity. In 1944, 88.64% of the principal and interest had been paid off. All the bonds were retired in 1944, when the contingent fund became available. The Act provided that 10% of the levy should be kept in a contingent fund to meet defaults.

The only levy now spread is for maintenance. In 1947 the amount was $585,893.

Essex Border Utilities Commission

This Act, as amended, is No. 15, of the year 1921, in the Province of Ontario. The Essex Border Utilities Commission was an interesting development, providing the necessary machinery for the handling not only of the sewers and sewage treatment, but also water works, board of health, hospitals, parks, and town planning, for a number of communities. As originally constituted, the Commission consisted of the Mayors of five towns, two reeves and ten members elected every three years. Money was raised by special taxes and by the sale of debentures, the term of which did not exceed thirty years. The cost of construction, maintenance, and operation was divided in proportion to the benefit, as determined by the engineer of the Commissioner. Questions were submitted to qualified voters for referendum and required the approval of at least three of the municipalities.

Under the Statutes of the Province of Ontario, the City of Windsor Amalgamation Act, 1935, dissolved the Essex Border Utilities Commission on July 1, 1935, and in place thereof substituted The Windsor Utilities Commission. There are three Divisions of this Commission: Hydro, Water, and General. Hydro and Water are the combined operations of the former

Windsor, Walkerville-East Windsor and Sandwich Water and Hydro Commissions, amalgamated under the above named Act. General assumed the former functions of the Old Essex Border Utilities Commission, namely, Filtration Plant operations, Westerly Distribution, Main Maintenance, Intercepting Sewer Maintenance, and House Numbering and Street Signs. The original costs of these undertakings by the former Essex Border Utilities Commission were assessed against the various municipalities amalgamated under the City of Windsor Act; also some outside municipalities to whom water is supplied.

The Metropolitan Sanitary District of Greater Chicago

The original Act of May 29, 1889 by the Illinois General Assembly, under which the Chicago Sanitary District operates, has been amended almost biennially, both to extend the powers of the District and to cure various defects found, as well as to provide for annexation of additional areas. Although originally drawn as a General Act, it has been adopted only by the Chicago Sanitary District. Powers are granted by the charter to build channels, drains, and docks, develop water power, and, by amendment, specific power to construct intercepting sewers, pumping stations, and sewage treatment Works. For the purpose of administration, a Board of nine members is provided, three of whom are elected every two years, and one of whom is chosen by the Board to act as presiding officer. The funds for construction are provided by the issuing of bonds, in amounts not to exceed 5% of the assessed valuation. For operating expenses, payment of interest, and the like, provision was originally made for a taxing power not to exceed 2/3 of 1% of the assessed valuation. Prior to 1940, the corporate fund of the Sanitary District had a maximum rate of tax of 15 cents per $100.00 of assessed valuation.

Since January 1, 1940, the Sanitary District has been operating under pegged levies for the corporate fund, which have gradually increased from $5,000,000 in 1940 to $20,000,000 in 1956, with an actual levy increasing from $5,000,000 in 1940 to $17,640,000 in 1957. The pegged levy statute requires the reduction of the levy from the amount authorized ($20,000,000 in 1957) for the collection of levies for prior years in the excess of amount required to retire tax anticipation warrants and interest thereon. This, in effect, limits the levy ($20,000,000) under average conditions to an actual amount of about $17,500,000.

Since 1952, no bonds have been issued for new construction purposes, funds for construction being obtained by levies of taxes amounting to: $5,600,000 in 1954, $5,600,000 in 1955, $9,200,000 in 1956, and $9,020,000 in 1957.

Organized in 1889 as The Sanitary District of Chicago with an area of 185 sq. mi. and a population of about 1,100,000, it now (1957) contains 920 sq. mi. and a human population of 4,815,000 (1956 estimate). The City of Chicago and 106 suburban cities and villages are within its boundaries. The name was changed to The Metropolitan Sanitary District of Greater Chicago effective October 13, 1955. The largest single addition, an area of about 400 sq. mi., resulted from a referendum vote November 6, 1956.

The original plan was to divert water from Lake Michigan to dilute raw sewage with 3-1/3 c.f.s. per 1,000 population. Owing to the decree of the U. S. Supreme Court (April 19, 1930), the diversion was ordered reduced to 1,500 c.f.s. on December 31, 1938. Consequently, sewage treatment works were built, with a comprehensive system of intercepting sewers, so that by

1950 sewage treatment works were in operation, with a combined capacity of 1,286 m.g.d. Since December, 1949, practically all the sewage has been given activated sludge treatment.

Expenditures for construction by the Chicago Sanitary District to January 1, 1957 are as follows:

Sewage Treatment Works	$146,033,023
Intercepting and Outfall Sewers	167,340,094
Sewage Pumping Stations	15,321,923
Sub-total for Sewage Treatment	328,695,040
Canals, River Improvements, Bridges and Appurtenances	86,205,194
Hydro-electric System	7,203,134
Administration Building	2,478,152
Grand Total	$424,581,420

Illinois Sanitary Districts

The difficulty of financing sewage disposal projects for the smaller cities of Illinois, ranging from 5,000 to 70,000 in population, led to a desire for a Sanitary District Act, which would provide the necessary funds. The 1889 Act creating the Sanitary District of Chicago did not prove applicable.

A total of five Acts are in existence in the laws of the State, authorizing Sanitary Districts, with broad powers:

Special Act of 1889
Cf. 1951 Illinois Revised Statutes, Chapter 42, Article 320 et. seq. and amendments thereto.

1907 Act
East Side Levee and Sanitary District (East St. Louis) Cf. 1951 Illinois Revised Statutes, Chapter 42, Article 247 et. seq., and amendments thereto.

1911 Special Act
Cf. 1951 Illinois Revised Statutes, Chapter 42, Article 277 et seq.

1917 Act
Cf. 1951 Illinois Revised Statutes, Chapter 42, Article 299 et seq.

1941 Act (for Sanitary Districts under 500,000 population)
Cf. 1951 Illinois Revised Statutes, Chapter 42, Article 319.1 et seq.

There is also an Act with very restricted powers:

1936 Act
Cf. 1951 Illinois Revised Statutes, Chapter 42, Article 412 et seq.

This covers sanitary districts and sewage disposal outside municipalities.

Sanitary Districts in the State of Illinois

In 1953, the following list shows all the Sanitary Districts known to the State Sanitary Water Board:

Special Act of 1889
The Sanitary District of Chicago

Act of 1907 (Chap. 42, Art. 247, Ill. Rev. Stat.)

East Side Levee and Sanitary District
(East St. Louis)

Special Act of 1911
North Shore Sanitary District.

Act of 1917
- Aurora (1925)
- Beardstown (1927)
- Bloomington-Normal (1919)
- Bloom Township (1928)
- Clinton (1925)
- Danville (1935)
- Decatur (1917)
- DeKalb (1928)
- Downers Grove (1921)
- East Peoria (1928)
- Elgin (1922)
- El Paso (1919)
- Galesburg (1924)
- Griggsville (1940)
- Hinsdale (1926)
- Indian Boundary (1927)
- Lincoln (1935)
- Lincoln Highway (1925)
- Marshall
- Norris City (1949)
- Peoria (1927)
- Rockford (1926)
- Round Lake (1948)
- Salt Creek (1928)
- Springfield (1924)
- Taylorville (1923)
- Urbana-Champaign (1921)
- Virden (1940)
- Wheaton (1924)

Acts of 1936
- Swissville (1937)
- South Roxana (1947)
- Central Stickney
- South Stickney
- Glen Oaks Acres
- Norwood Park
- Orchard Place
- Barrington Woods

The Act of 1936 authorizes the formation of sanitary districts for local service in building local sewer systems, and may be employed within a larger district organized under a different act, e.g., within The Sanitary District of Chicago are two sub-districts in recently developed areas, known as the Central Stickney and the South Stickney Sanitary Districts.

The 1911 Act was drawn largely with reference to villages taking water from Lake Michigan. Under this Act, 300 legal voters may petition the County

Judge of the County, thereby starting the machinery for an election for the formation of the proposed Sanitary District. If formed, provision is made for a Board of five Trustees (each receiving not over $1,000.00 per annum), who are appointed by the County Judge and two Circuit Judges. General powers are given to the Sanitary District to construct works, and, further, levy an annual tax for corporate expenses not to exceed 0.083 per cent of the assessed valuation. Before any bonds can be issued, a referendum has to be held. The amount of bonds shall not exceed 5% of the valuation. Under this Act, only one Sanitary District has been incorporated—The North Shore Sanitary District.

Under the Act of 1917, twenty-nine Sanitary Districts have been incorporated, the largest being the Greater Peoria Sanitary District (1950 population, about 111,865). The general provisions resemble those of the Act of 1911, except that 100 legal voters may petition for an election for the formation of the District, and, further, that the Board of Trustees consist of three members (each receiving not over $1,000.00 per annum), appointed by the County Judge. Broad powers are given to the Board for the purposes of constructing sewerage works and sewage treatment works. Money may be raised by bond issues to mature in not exceeding twenty annual installments; the amount of the bond issues not to exceed 5% of the valuation. Provision is made for levying taxes to pay interest and annual installments, as well as for taxes for corporate purposes in amount not to exceed 0.083 per cent of the assessed valuation, providing, however, that an additional 0.083 per cent may be authorized by referendum.

Wisconsin

The Milwaukee Sewerage Commission originally operated under an Act entitled "An Act Relating to the Sewage Disposal Works in Cities of the First Class," being the following chapters of the Laws of Wisconsin:

608 of 1913; 328 of 1915; 607 of 1915; 304 of 1917; and 657 of 1919.

This Act provides that the common council of any city of the first class may vote for the establishment of a sewerage commission, and if carried by a majority, the Mayor appoints five members. These serve without pay. The Commission has power to construct intercepting sewers and sewage treatment works. For financing, a tax of one mill on the dollar may be raised. Bonds may be issued without referendum, unless petitioned by 15 per cent of the number of voters at the last preceding election. The bonds shall not exceed $10,000,000, or 5 per cent of the assessed valuation.

Provision was made in 1917 for approval of plans where other towns in the same county are doing similar work. This has been enlarged (in 1921) to provide for a Metropolitan Sewerage Commission, consisting of three members appointed by the Governor, one of whom is to be a member of the City sewerage commission. The work authorized will be financed by bonds, running twenty years, and after the first ten years to be redeemed in ten annual installments. Funds may also be raised for operations and maintenance by assessment on the cities and villages, to be raised by taxes. The income from the sale of heat-dried activated sludge helps to pay operating and maintenance.

In Wisconsin there is the Town Sanitary District Law, which permits the formation of sanitary districts for various purposes, including garbage disposal, sewage disposal, and water supply (Wisconsin Statutes, Chapter 60, Sections 60.30-60.309). This permits levying of taxes or collection of funds by sewer rentals or sewerage service charges.

In addition there is the Metropolitan Sewerage District's law, under which three districts have been formed: Milwaukee, Madison, and Green Bay. These have no right to supply water.

Greater Winnipeg Water District

The Greater Winnipeg Water District was organized under Chapter 22 of the Acts of 1913, Province of Manitoba, and provides for the organization of a municipal corporation, including, at first, four municipalities. The Board is made up of the Mayor and Board of Control of the City of Winnipeg; the Mayor and one member of the Council of the City of Boniface, and Mayor or Reeve, as the case may be, of the minor municipalities. The Mayor of Winnipeg is Chairman of the Board. The purpose of this Water District is to supply the inhabitants with pure water from a permanent source in bulk to each of the municipalities. The cost of service is to be arrived at on the basis of maintenance, operation, and management. One-half of the amount required annually to pay interest and sinking fund, maintenance, and operation, is to be borne by the land as a tax. The corporation is authorized to borrow and issue bonds with interest not exceeding 5 per cent per annum and a life not exceeding fifty years. The method of issuing bonds must be approved by referendum vote of the City of Winnipeg. The corporation is authorized to levy taxes to pay interest and a sinking fund to retire the bonds. The Act is very specific as to the financing, and, in particular, to the taxation, both as to the spreading of taxes and the exemptions.

New York State

The status of taxation in New York State is thoroughly discussed by D. T. Selko(2) in a special report of the State Tax Commission on Town-administered special districts and the control of local finance in New York, and also by P. E. Malone(3) in a special report of the State Tax Commission on the Fiscal Aspects of State and Local Relationships in New York. In the latter report is a brief discussion on Districts Authorized under Special Legislative Acts (pp. 132-136), such as the

Niagara Frontier Planning Board
Central New York and Lower Hudson Regional Markets,
Thousand Island Bridge Authority, and
Triborough Bridge Authority.

The affairs of these districts are managed by Boards appointed by cities and/or counties in the district. Operating revenues required by the market and bridge authorities are derived from fees and tolls charged to users of the facilities.

In New York State there are no specific districts which are classified as "Metropolitan." However, there are three counties in the State where sewerage and sewage treatment functions for the entire county are under the respective control and direction of the Nassau County Department of Public Works in Nassau County; the Westchester County Department of Public Works in Westchester County; and the Onondaga Public Works Commission in Onondaga County. There are about 150 sewer districts, varying from 200 to 35,000 population.

For the distribution of water, there are five such districts in the State; four including single counties, i.e., the Erie County Water Authority, Monroe

County Water Authority, Onondaga County Water Authority, and the Suffolk County Water Authority. The other water authority, the Northwestern New York Water Authority, includes Niagara and Orleans Counties.

Westchester County

The Westchester County Sanitary Sewer Commission was organized by the Board of Supervisors of Westchester, under Chapter 603, Laws 1906. It serves an area of about one hundred square miles, which was divided into six main projects. Primary treatment with sterilization was considered adequate.

Board of Water Supply, New York City

The Board of Water Supply, New York City, was authorized by Chapter 724 of the Laws of 1905, New York State.

For many years the City of New York constructed large public improvements under the auspices of a special Commission or Board created for that purpose, having broader powers than possessed by any of the City departments. So the Act was passed, with the object of securing administrative authority with broad powers and not subject to the many annoyances and delays to secure successive approval of the public bodies, as in the case of the regular City departments, and also to secure reasonable permanence in office. The Act provides that a Commission, to be known as the Board of Water Supply, with three members, be appointed by the Mayor, and further provided that "no member of said board shall be removed except for incompetency or misconduct shown after a hearing upon due notice, upon stated charges", this Board to determine what sources were most available, desirable, and best for an additional supply of pure and wholesome water for the City of New York, and report to the governing Board of the City, the Board of Estimate and Apportionment. This Board was to approve the outlined plan of the project for a new water supply, which could be submitted in parts or as a whole and be modified later, if desirable, with the approval of the same body. It was the duty of the Board of Water Supply to prepare contracts and carry out the work without any further approval of the governing Board of the City.

The Board of Water Supply, further, has authority to appoint all necessary employees and to fix their compensation, subject to Civil Service rules.

The necessary funds were authorized from time to time by the governing Board of the City, a strict accounting of which was kept by the auditing department, and frequently reports were made to the Comptroller and to the City authorities. The City of New York does not actually raise the funds and deposit them, as is the case in some cities, but it issues its corporate stock to take care of capital expenditures as needed from time to time.

The contracts receive only the approval as to form of the Corporation Counsel, and the Board has full authority to award contracts to other than the low bidder. With reference to this, the Act provides that the Board may select the bid or proposal which will in their judgment best secure the efficient performance of the work, and provides, further, that "no contract shall take effect until the Board of Water Supply or a majority thereof shall certify thereon in writing that its acceptance will in their judgment best secure the public interest and the efficient performance of the work therein mentioned." This is a privilege which has been very seldom exercised, but, nevertheless, has proved very valuable in some instances.

A great deal of the Act is taken up with the procedure for the condemnation of real estate and trial of the cases, which is the same procedure that has been used by the City for a great many years; in fact, this whole Act is practically a repetition of the Act under which the Aqueduct Commissioners operated between 1883 and the time of the abolition of that Commission in 1910.

Ohio

In Ohio the only district of a metropolitan type is the Mahoning Valley Sanitary District, which provides a water supply for Youngstown and Niles and some suburban areas. Under the Ohio law, only representatives of the two cities which organized the District control its operations.

Around the large cities in Ohio, such as Cleveland, Cincinnati, Columbus, Toledo, Akron, Dayton, and Canton, there are "County sewer districts." In most cases, the water supply is obtained from the large city and the sewers in the surrounding areas discharge into the sewerage system of the large city. The County Commissioners make a contract with the City for sewerage and water service to the County sewer district.

Pennsylvania

Pennsylvania is one of the states which have enacted general permissive legislation allowing the formation of special non-taxing districts (authorities) to finance revenue-producing projects. The original Act was passed in 1935 (known as the Municipal Authorities Act of 1935) and subsequently repealed in 1945, and replaced by the Municipal Authorities Act of 1945, passed on May 2, 1945 and subsequently amended on June 12, 1947. The municipal authority is a device whereby municipalities can acquire and operate revenue-producing projects, otherwise not available to them because of constitutional debt limits. An Authority is a special public corporation whose obligations are payable solely from its revenues. It lacks taxing power, and by statute must be self-liquidating. Its corporate existence is limited to fifty years. The law specifically limits its functions (Section 4). An Authority is administered by a five-man board, generally serving without compensation.

The most popular types of Authority projects in Pennsylvania are water works, public school buildings, and sewage disposal Authorities. Others now operated are parking lots, garages, airports, office and factory buildings, flood control facilities, arenas, and traffic control centers. The total number of municipal authority projects in operation on January 1, 1951, was 232 (City Planning Advisory Board, Bull. 18, May, 1952).

As of June 1, 1952, the records(4) in the office of the Secretary of the Commonwealth showed that 516 municipal authorities had been created and established in Pennsylvania. Of these, some 300 were active (including 105 water, 79 school, 67 sewer, 11 airport, 10 parking, 22 building, and 6 other).

In the field of Sewerage and sewage disposal, the outstanding example is Allegheny County, of which Pittsburgh is the center, wherein there has been established the Allegheny County Sanitary Authority, operated by a Board of five members. The original plan of the Authority was to collect and treat the sewage of some one hundred municipalities in Allegheny County and also some two or three in adjoining Westmoreland County. This plan has since been modified to care for the sewage of Pittsburgh and some other sixty municipalities in the center of the County. The remainder of the municipalities have been grouped into six or seven district areas comprising from two to three

towns to perhaps a dozen, which will collect and treat their sewage and certain industrial wastes in separate plants. In the neighborhood of Philadelphia, some twenty-nine municipalities are segregated into three distinct districts, as follows:

Central Delaware Authority	12
Muckinipatis Authority	8
Darby Creek Joint Authority	9

In the water works field, no metropolitan plan has been established.

Standard Metropolitan Areas

The United States Bureau of the Census defines a standard metropolitan area as one containing at least one city of 50,000 population or more in 1950, and each city of this size is included in one standard metropolitan area. In general, each standard metropolitan area comprises the County containing the city and any other contiguous counties which are deemed to be closely and economically integrated with that city. Such areas in 1950 ranged in size from the New York-Northeastern New Jersey area, with a population of 12,911,994, to the Laredo area, with a population of 56,414. There were 168 standard metropolitan areas in the continental United States.

In the classification by the Census, the following data are given for 1950:

	Population 1950	Per Cent Increase 1940	1950
New York-Northeastern, N. J.	12,911,994	10.7	
Chicago, Ill.	5,495,364	13.9	
Los Angeles, Calif.	4,367,911	49.8	
Philadelphia, Pa.	3,671,048	14.7	
Detroit, Mich.	3,016,197	26.9	
Boston, Mass.	2,369,986	8.8	
San Francisco-Oakland, Cal.	2,240,767	53.3	
Pittsburgh, Pa.	2,213,236	6.3	
Cleveland, Ohio	1,465,511	15.6	
St. Louis, Mo.	1,681,281	17.4	
Washington, D. C.	1,464,089	51.3	
Baltimore, Md.	1,337,373	23.5	
Minneapolis-St. Paul, Minn.	1,116,509	18.7	
Buffalo, N. Y.	1,089,230	13.6	

Compacts

On June 30, 1948, the fifth Interstate Pollution Control compact in the United States went into effect between Illinois, Indiana, Kentucky, New York, Ohio, Pennsylvania, Virginia, and West Virginia, replacing an informal

agreement made in 1928 between the same eight states and Maryland, Tennessee, and North Carolina. This Compact is administered by the representatives of these states as the Ohio River Valley Water Sanitation Commission. The Interstate Sanitation Commission (1936), the Interstate Commission on the Delaware River Basin (1936), the Interstate Commission on the Potomac River Basin (1941) and the New England Interstate Water Pollution Control Commission (1947) were created by earlier compacts. The Ohio compact covers the greatest number of states, the greatest drainage area (155,000 sq. mi.) and the largest population, in 1950 (17,600,000). These have been summarized by K. S. Watson[5] as shown in Table 1.

Three interstate pollution control agreements are in force as of September 1, 1948, known as the Great Lakes Drainage Basin Sanitation Agreement, the Upper Mississippi River Drainage Basin Sanitation Agreement, and the Interstate Sanitation Committee (North Dakota, South Dakota, and Minnesota). These have been summarized by K. S. Watson[5] as shown in Table 2.

The Interstate Sanitation Commission is the second largest pollution control organization operating under a compact within the States of New York, New Jersey, and Connecticut. In 1952, the sewage treatment works under its supervision had a capacity of 314 mgd. Eventually the capacity will be increased to 850 mgd. Under construction are plants with a capacity of 80 mgd, to be completed in 1953.

Its industrial waste inventory prior to 1952 included some 1,500 plants, of which some 560 generated water-borne wastes. Eventually some 27,000 plants are to be visited.

Bi-State Metropolitan Development District

On July 26, 1949, the Illinois General Assembly authorized a compact with Missouri to facilitate the future planning and development of the Bi-State Metropolitan District, which includes the City of St. Louis, and St. Charles and Jefferson County in Missouri, and the Counties of Madison, St. Clair, and Monroe in Illinois. The District is authorized to plan, construct, maintain, and operate bridges, tunnels, airports, and terminal facilities, and plan and establish policies for sewage, drainage, and other facilities. The District may issue revenue bonds for the construction of any facility or for acquiring any facility which it intends to operate. It has no taxing powers. The States of Missouri and Illinois have appropriated about $25,000 each year for current expenses in maintaining an office and preparing programs for review.

ACKNOWLEDGMENTS

This paper was prepared by the late Langdon Pearse, M. ASCE, in 1953. His acknowledgments, in addition to the references, were "to the following gentlemen for information kindly furnished, some of which has been used verbatim: J. R. Downs, R. H. Gould, Glenn D. Holmes, A. E. Morgan, Asa E. Philips, J. Waldo Smith, and A. B. Wood."

The paper was submitted to the Society for publication by Norval E. Anderson, M. ASCE,[6] acting as proxy. The proxy has brought up to date some of the data, principally regarding The New York Port Authority, The Metropolitan Sanitary District of Greater Chicago, Table 1, and Table 2. The proxy will undertake to answer any discussions.

REFERENCES

1. "The Government of Metropolitan Areas," prepared by Paul Studenski and the Committee on Metropolitan Government (1930) National Municipal League, New York City.

2. "Town Administered Special Districts and the Control of Local Finance in New York," by Daniel T. Selko; Special Report of the State Tax Commission No. 11 (1936), Albany, N. Y.

3. "The Fiscal Aspects of State and Local Relationships in New York," by Paul E. Malone; Special Report of the State Tax Commission No. 13 (1937), Albany, N. Y.

4. "Water Authority in Pennsylvania," by S. C. Braucher, C. N. Fisher, I. S. Sahli, and J. C. DeGroot; Journal, American Water Works Association, February, 1953, Vol. 45, pp. 153-169.

5. "Ohio River Compact and other Interstate Agreements," by Kenneth S. Watson; Journal American Water Works Association, January, 1949, Vol. 41, pp. 18-22.

6. Engineer of Treatment Plant Design. The Metropolitan Sanitary District of Greater Chicago.

Table 1

INTERSTATE POLLUTION CONTROL COMPACTS (1957)

ITEM	OHIO RIVER VALLEY WATER SANITATION COMMISSION	INTERSTATE COMMISSION ON POTOMAC RIVER BASIN
Signatory States	Ind.,W. Va., Pa., Ill., N.Y., Ohio, Ky., Va. (8)	Md. W. Va., Va., Pa., D.C. (5)
Effective Date	1948	1941
Commissioners per State	3	3
Area - sq.mi. *	154,880	14,500
Population *	15,822,337	2,100,000
Functions	Issue orders regulating pollution.	Coordinate; investigate, summarize, technical data, act as advisory agency; promote uniform legislation and practices.
Minimum Treatment Requirements	Removal of not less than 45 per cent of suspended solids.	None specified in compact; commission directed to recommend standards.
Stream Zoning	None specified.	Preliminary zoning complete.
Special Features	Consent of Congress given for other Ohio River states to become members.	

* As of September, 1948.

Table 1 (Continued)

INTERSTATE POLLUTION CONTROL COMPACTS (1957)

INTERSTATE SANITATION COMMISSION	INTERSTATE COMMISSION ON DELAWARE RIVER BASIN	NEW ENGLAND INTERSTATE WATER POLLUTION CONTROL COMMISSION
N.Y., N. J., Conn. (3)	Del., N.Y., Pa., N.J. (4)	Conn., R.I., Mass. (3) **
1936	1936	1947
5	5	5
	13,000	13,926
11,000,000	5,000,000	6,739,309
Substantially same as Ohio River Compact.	Coordinate enactment of uniform state laws; develop and propose objectives; states exercise enforcement authority.	Coordinate and advise; adopt water standards; classify streams; states exercise enforcement authority.
Two classes of treatment specified, depending upon zone of carrying stream.	Waste specifications based on zone of carrying stream.	None specified.
Class A and B zones specified in compact.	Compact specifies four zones.	Compact authorizes zoning; tentative standards adopted.
	Program effectuated by terms of reciprocal legislative agreements not requiring ratification by Congress.	Compact does not pertain to one basin but to all interstate waters among various states.

** Eligible to join: Me., N.H., Vt., N.Y.

Table 2

MAJOR INTERSTATE POLLUTION CONTROL AGREEMENTS (1957)

ITEM	GREAT LAKES DRAINAGE BASIN SANITATION AGREEMENT	UPPER MISSISSIPPI RIVER DRAINAGE BASIN SANITATION AGREEMENT	INTERSTATE SANITATION COMMITTEE
Signatory States	Wis., Minn., Ill., Mich., Ohio, Pa., N.Y., Ind. (8)	Minn., Iowa Wis., Ill., Mo. (5)	N.D., S.D., Minn. (3)
Effective Date,	1928	1935	1944
Cooperating Stage Agency	Dept. of Health and Pollution Control Agency	Dept. of Health through Sanitary Engineering Division	Dept. of Health and Conservation Commission
Minimum Treatment Requirements	Primary for sewage	None specified	25 ppm B.O.D., 30 ppm Suspended Solids
Stream Zoning	None	None	None
Special Features	Annual two-day meetings only	Agreement outgrowth of interstate development between Minnesota and Wisconsin in 1925	

Paper 1681 June, 1958 SA 3

Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

EFFECTS OF AERATION PERIOD ON MODIFIED AERATION[a]

Wilbur N. Torpey,* M. ASCE and Martin Lang,** M. ASCE
(Proc. Paper 1681)

New York City has been faced with special problems in treating sewage because of its location on the various estuaries that make up New York Harbor. Although considerable tidal flows are available for dilution of sewage effluent, the oscillatory motion in the deep channels of the harbor creates localized areas where it is difficult to secure satisfactory concentrations of dissolved oxygen.

Plain sedimentation of sewage, with its limited capacity for removal of B.O.D., was incapable of meeting the requirements of the situation. On the other hand, conventional activated sludge plants, with their high removal efficiency, required considerably greater costs. New York City was impelled, therefore, to develop modifications of the activated sludge process to economically relieve the pollutional load on the harbor.

One of these processes, "Modified Aeration," developed by L.R. Setter,(1,2) has been employed successfully since 1943. This process is used to treat either raw sewage or settled sewage in tanks providing aeration periods of about 2 hours, ranging from 1.4 to 3 hours. Treatment efficiency, ranging from 60 to 80 per cent of removal of B.O.D., depends largely upon the particular sewage characteristics.

Until the Owls Head plant was built, limited hydraulic facilities of existing aerators prevented the exploration of aeration periods much below 2 hours. In this plant the flexible layout of the channels and aerators made it possible to operate at higher aerator loading rates by successively removing aerators from service. The results of such operation, using aeration periods ranging from 2.7 to 0.5 hours, are presented herewith.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1681 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

a. Presented at meeting of ASCE, New York, N. Y., October, 1957.

* Senior Civ. Engr., Bureau of Sewage Disposal Operation, Dept. of Public Works, New York City.

** Civil Engr., Bureau of Sewage Disposal Operation, Dept. of Public Works, New York City.

Description of the Owls Head Plant

The Owls Head plant was designed to use the modified aeration process, and has been described by Steffensen.(3) This plant, with an ultimate capacity of 160 M.G.D., provided for the aeration of raw sewage in the presence of return sludge. The layout of the aeration and final tanks is shown in Figure 1. Raw sewage and return sludge enter the aerated influent channel forming mixed liquor which may be admitted to any of the six aeration tanks via the 60-inch diameter inlet pipes, or may be conducted directly to the aerated effluent channel through two 7' x 7' bypass gates. The aerated influent and effluent channels were designed as part of the aeration tank system. The six aeration tanks have a total capacity of approximately 1,500,000 cubic feet, and the channels 380,000 cubic feet. Maximum tank and channel width is 25 feet. Air is diffused by ceramic tubes supported on retractable headers.

The plant is equipped with eight rectangular settling tanks, each 300 feet long by 52 feet wide by 12 feet water depth. Aerator effluent is admitted to one end of the tank and the plant effluent overflows longitudinal weirs occupying the far quarter of the tank. Sludge is raked by a double collector mechanism to a removal sump at the center of the tank. Each tank sump is connected to both a centrifugal pump and a plunger pump, so that any tank can be used for either returning sludge to the aerators or for thickening.

First Four Years of Operation

The plant was placed in service in February 1952, using the modified aeration process. The sewage flow increased to about 80 M.G.D. in April 1952 and has since gradually increased to about 87 M.G.D. Based on experience in other New York City plants, the aeration period was initially set somewhat about 2 hours, while final tank overflow rates were adjusted to about 800 gallons per square foot per day. The aeration period and the final tank overflow rate were determined by the number of units in service.

The process was controlled to reach for the highest sludge age consistent with maintaining satisfactory sludge settling characteristics. The sludge age was a function of the volume of aeration tanks in service and of the suspended solids concentration of the aeration liquor, relative to the daily raw sewage suspended solids load. The aeration liquor suspended solids concentration was governed by the ratio of the solids returned to the aeration tanks versus the solids removed from the process by those final settling tanks used as thickeners. At some times falling sludge density index and increasing populations of sphaerotilus indicated that the sludge age was too high for the particular conditions of operation. The age was then reduced by changing the function of one of the settling tanks from that of returning sludge to that of thickening sludge. On the other hand, as the sludge age was lowered below the optimum, the treatment efficiency tended to approach that of plain aeration. Changes in age were required mainly by seasonal variations in the temperature and strength of the sewage.

The air supply was adjusted to maintain about 2 p.p.m. of dissolved oxygen in the aerator effluent entering the first final tank. For the first four years of operation the aeration system consisted of one, two, three or four aeration tanks, in addition to the aerated channels, and the plant effluent overflowed seven final settling tanks. The data derived from these four years of operation

employing the methods of control outlined above are presented in the form of annual averages in Table I.

During this time an average of 83 M.G.D. of sewage was treated by the modified aeration process. With 2.2 hours of aeration and an aerator suspended solids concentration of 360 p.p.m., the sludge age averaged 0.21 day. The return sludge rate averaged 12 per cent of the sewage flow, and 0.47 cu. ft. of air was required per gallon of sewage treated. At a final settling and sludge thickening tank overflow rate of 780 gallons per square foot per day, the plant effluent contained 46 p.p.m. of suspended solids and 67 p.p.m. of B.O.D. The percentage of suspended solids and B.O.D. removed was 73 and 60 per cent, respectively. One noteworthy aspect of these results is the uniformity of these yearly averages.

Estimated Performance of Plain Sedimentation

It would be of interest to determine what the treatment efficiency would have been without the benefit of aeration, but it was not possible to bypass the aeration system of this plant. However, by the use of the "filtrate" B.O.D. data in Table I, the B.O.D. removal efficiency of plain sedimentation at Owls Head may be estimated with reasonable accuracy.

This calculation is based on the assumption that the Owls Head final settling tanks by themselves would remove 60 per cent of the suspended solids in the raw sewage and would not affect the filtrate B.O.D. The filtrate B.O.D. was determined as the B.O.D. of the liquor after passing through a suitable filtering medium to remove the suspended material. The B.O.D. of the plain sedimentation effluent would then include this raw sewage filtrate B.O.D. as well as the B.O.D. due to the remaining suspended solids. These solids should exert the same B.O.D. per part of suspended solids as those in the modified aeration effluent. For the period of 1952 through 1955, the final effluent suspended solids of 46 p.p.m. exerted 67 minus 39, or 28 p.p.m. of B.O.D.

For the data of Table I, a plain sedimentation removal efficiency of 60 per cent would yield an effluent containing 67 p.p.m. of suspended solids. This effluent would contain 71 p.p.m. of filtrate B.O.D., plus 41 p.p.m. of B.O.D. attributable to 67 p.p.m. of suspended solids, for a total of 112 p.p.m. of B.O.D. Since the raw sewage B.O.D. was 166 p.p.m., plain sedimentation would have removed 33 per cent of the B.O.D., which conforms to experience. The use of the aeration system thus decreased the B.O.D. in the effluent from the 112 p.p.m. for plain sedimentation to 67 p.p.m. for modified aeration.

Included in Table I are times when from one to four aeration tanks were used. These annual averages include aeration periods of from 1.1 to 2.7 hours. Operation at 1.1 hours was found to yield treatment results closely approximating those obtained using longer detention periods.

Preparation for Low Aeration Period Operation

It was therefore decided to investigate the effects of decreasing the aeration period to the lowest possible at the Owls Head plant. This was accomplished by removing all aeration tanks from service and opening the bypass gates between the aerator influent and effluent channels, so forming a

continuous aerator. This "channel aerator" conducted the liquor to the final settling tanks along the course of its flow, as shown in Figure 1.

The aeration period varied, when using the channel aerator, for the liquor entering the individual final tanks, increasing rapidly toward the last unit. The average detention time was calculated on the basis of an aerator volume of 306,000 cu. ft., corresponding to the channel aerator capacity to the mid-point of the final settling tanks. For average flow, this capacity provided an aeration period of 33 minutes. At the first final settling tank the liquor had been in the aerator only 20 minutes. At the last settling tank the remaining one-seventh of the liquor had been in contact with the modified aeration sludge for about 60 minutes.

In order to obtain the higher solids concentration necessary to maintain the desired sludge age in this 33-minute aerator, only one final tank was used as a thickener, and the flow of aerator effluent to this final tank was throttled. The other six final tanks were used for returning sludge. Additional tubes were installed on the air diffuser manifolds, doubling the number previously in service, to maintain normal tube loading rates. The plant sampling schedule was augmented to include individual grab samples taken five times daily of the effluent from the first, middle and last final settling tanks. Otherwise, the plant was operated in essentially the same manner as before.

Low Aeration Period Operating Results

Operation at this low aeration period was started on February 15, 1956, and the necessary change-over was completed by March 1, 1956. Treatment results for the ensuing year, in the form of monthly averages, are presented in Table II.

Comparing this year with the previous four years of operation (Table I), it can be seen that for the first 10 months the flow increased significantly to 87 M.G.D. During the last two months of this period the flow was reduced by construction work in the drainage area, reducing the overall yearly average to 85 M.G.D. The air requirement of this 33-minute aerator was reduced to 0.24 cu. ft. per gallon from the 0.47 cu. ft. per gallon previously demanded, representing a saving of one half of the power for compressing air. Sludge age and final tank overflow rate were substantially the same. In order to maintain the same sludge age, the suspended solids concentration of the aerator effluent was increased to 920 p.p.m. from the previous 360 p.p.m. by using one thickening tank with throttled inflow.

The analytical results show that the final effluent contained 49 p.p.m. of suspended solids and 75 p.p.m. of B.O.D., as compared with 46 p.p.m. and 67 p.p.m., respectively, for the previous four years. The filtrate B.O.D. in the effluent was 45 p.p.m., as compared with 39 p.p.m. obtained previously.

Operating Factors

During the first four years of operation the return sludge pumps presented some difficulty because of clogging, especially during rainy periods. This clogging resulted from the fact that there were no preliminary sedimentation tanks to remove fibrous material which formed mats in these pumps. Operation of the plant at high aerator loading rates aggravated this problem in two ways, by having to place more return sludge pumps in service, and by tripling

the concentration of solids passing through them. Such clogging affected treatment results adversely. Thus, after the rainstorm of October 31, 1956, which resulted in an inrush of fallen leaves, the return sludge pumps were so badly affected that it required almost five days during November to resume normal operation.

Another difficulty arose from the much larger quantity of solids circulating through the final settling tanks. In attempting to operate at the highest possible sludge age, we risk, and sometimes encounter, the proliferation of filamentous organisms. During previous operation, this condition could be corrected by cautiously increasing the ratio of wasting to returning sludge. When such conditions were encountered in September and October, 1956, it took some time to realize that such measures were only temporarily effective. Because of the high aerator suspended solids concentration, so much solids were circulating through the final settling tanks as to require complete removal of the solids from the system on one day and restoring operation the following day.

The reduction in aerators to only the influent and effluent channel aerators resulted in high air diffusion rates per lineal foot of tank. Although the maintenance of adequate dissolved oxygen concentrations in the aerator presented no difficulty, yet the high rates of air application resulted in a marked increase in the volume of skimmings removed from the final settling tanks.

Effects of Aeration Period

Thus far there has been presented the results of the first four years of operation in Table I, and the data for a year of low aeration period operation in Table II. In order to determine the effects of the aeration period on treatment results, the data for these five years of operation were grouped by full months according to the number of aeration tanks in service. Such grouping should provide a valid comparison inasmuch as operation for the five years required practically the same sludge age regardless of the aeration period. These data, grouped according to number of aeration tanks in use, and hence according to aeration period, are shown in Table III.

These data show that sewage flow and final tank detention have been substantially uniform at Owls Head. The aeration period varied from 2.7 to 0.55 hours. The sludge age centered around 0.20 day. The aerator effluent suspended solids concentration varied from 310 p.p.m. for 2.7 hours aeration period to 920 p.p.m. for 0.55 hour. Since higher solids concentrations required that more of the final tanks be used for returning sludge, the return sludge rate increased from 11 to 19 per cent of the sewage flow as the aeration period was decreased.

For aeration periods of 1.7 to 2.7 hours, the quantity of air required approximated 0.5 cu. ft. per gallon of sewage. At 1.1 hours the air ratio was lowered to 0.43. At 0.55 hour there was a substantial decrease to 0.24 cu. ft. per gallon, while maintaining aerator effluent dissolved oxygen concentrations comparable to previous operation.

Suspended solids removal was virtually unchanged from 1.1 to 2.7 hours, averaging 73 per cent, but there was a decrease to 68 per cent at 0.55 hour. Similarly, B.O.D. removal was in the order of 60 per cent for aeration periods ranging from 1.1 to 2.7 hours. At 0.55 hour, however, B.O.D. removal decreased to 53 per cent.

The data of Table III were plotted graphically to aid in determining the curves presented in Figure 2. The position and direction of these removal curves in the zone of changing curvature could not be precisely defined from the data of Table III alone.

Starting May 1956, samples were taken of the effluent of the first, middle and last final settling tanks along the aerator channel. These samples correspond to aeration periods of 20, 33 and 60 minutes, respectively. These samples were collected manually five times daily at such intervals as to approximate equal volumes of tank effluent between samples. These data are presented in Table IV, which denotes the concentration of suspended solids, B.O.D. and filtrate B.O.D. in the effluent from final tanks 1, 4 and 8, corresponding to detention times of 20, 33 and 60 minutes. The validity of this type of sampling was established by the fact that the overall averages of the three tank samples corresponded with those secured by continuous automatic sampling, the results of which were used in the previous tables.

These data were then used to help establish the position of the removal curves of Figure 2 in the region of lower aeration period. The removal efficiency at each of these aeration periods have been placed on Figure 2 and used to extend the solid lines to an aeration period of 0.33 hour.

The curve for suspended solids removal shows that lowering the aeration period from 2.7 to 0.33 hours resulted in a gradual decrease of removal efficiency from 74 to 68 per cent. The curve for B.O.D. removal indicates that the efficiency was not affected by lowering the detention time from 2.7 hours to one hour, averaging 60 per cent over this interval, but thereafter decreased to 50 per cent at 0.33 hour.

These curves were then completed by dashed lines to zero aeration period, corresponding to plain sedimentation which would yield about 60 per cent removal of suspended solids, based on data from other New York City plants. The B.O.D. removal efficiency was previously calculated to be 33 per cent for plain sedimentation.

The air ratios at various detentions have also been plotted on Figure 2. It should be noted that the air requirements decreased slowly as the aeration period was lowered to one hour, but thereafter decreased sharply and much more rapidly than did the B.O.D. removal. Thus, from the curves, at 1.0 hour of aerator detention 0.40 cubic foot of air per gallon was needed to remove 60 per cent of B.O.D., while at 0.55 hour when the B.O.D. removal was 53 per cent, the air required was 0.24 cubic foot per gallon. Because of this marked reduction in air requirements, no difficulty was encountered in diffusing sufficient air to maintain the desired dissolved oxygen concentration in the aeration liquor of this highly loaded aerator.

In referring to the removal of B.O.D. by aeration processes, it is important to denote the loading rate of the aeration tanks. This loading rate is expressed as lbs. of B.O.D. per day per 1000 cu. ft. of aerator volume. From Table III it can be seen that the critical one hour detention time, below which the B.O.D. removal decreased significantly, corresponded to a loading rate of 180 lbs. per day per 1000 cu. ft. based on raw sewage B.O.D. Furthermore, at loading rates of 370 lbs., for 33-minute aeration period, much useful biological work was still accomplished.

Discussion of Results

The above data represent five years of continuous use of the modified aeration process, treating 85 M.G.D. of raw sewage at the Owls Head plant. During this time most operating factors were kept constant. Sufficient air was supplied to maintain about 2 p.p.m. of dissolved oxygen in the aerator effluent during the day. Since the sewage flow was relatively uniform, final tank overflow rates were substantially constant. During this time the age of the sludge maintained in the aeration tanks scarcely changed from 0.20 day, inasmuch as any significant increase over this value soon resulted in deterioration of settling qualities of the floc. The changes in aerator effluent suspended solids concentration merely reflected the adjustments required to maintain a constant total weight of solids in the aeration tanks. Sewage strength fluctuated only slightly over the course of these five years.

The only significant variation in operation was in the number of aeration tanks used. Depending on the number of units in service, the aeration period varied in steps from 2.7 to 0.55 hours. The corresponding range of B.O.D. loading rate was from 90 to 370 lbs. per day per 1000 cu. ft. of tank.

During the first four years of operation, which includes a variation in aeration period from 1.1 to 2.7 hours, there was no marked change in treatment efficiency. In other words, changes in aerator loading rate of from 90 to 180 lbs. of raw sewage B.O.D. did not demonstrate any significant differences in suspended solids and B.O.D. removal efficiency. However, lowering of the aeration period did result in some decrease in the demands for compressed air.

On the other hand, as the aeration period decreased below one hour, the removal of B.O.D. and the air demands fell off sharply, whereas suspended solids removal was not greatly influenced until the aeration period was reduced below 20 minutes. For the year of operation at an aeration period of 33 minutes and an aerator loading rate of 370 lbs., suspended solids removal averaged 68 per cent, B.O.D. removal averaged 53 per cent and air requirements were 0.24 cu. ft. per gallon. The corresponding values for the previous four years at 2.2 hours were 73 per cent, 60 per cent and 0.47, respectively.

New York City is presently operating five modified aeration treatment plants. It might appear that these plants should have almost identical sewages and operating results. However, even within the City of New York itself there are major differences in sewage characteristics and process performance. Some sewages are relatively insensitive to variations in sludge age, and operation is readily controllable over a wide range of conditions. With such sewages higher sludge ages can be used; for example, at the Jamaica plant, where but 1.5 hours of aeration will effect removals of 75 per cent. Although the Owls Head sewage is of domestic origin and of medium strength, it has proved the least amenable to treatment by modified aeration. Therefore, it is reasonable to expect that at equal aeration periods these results can be duplicated or improved when applied to other plants within New York City.

However, the Owls Head experience should not be used to predict the efficiency of treatment of sewages having materially different strengths or temperatures, or containing high proportions of industrial wastes. When such different sewages are to be treated by modified aeration, a specific investigation should be made to determine the treatability and operating factors.

Although individual plants will differ, it may be assumed that the performance of each plant will generally conform to the behavior patterns defined in the curves of Figure 2. From these curves it seems plain that at a constant sludge age, as the aeration period increases, an aerator volume is reached beyond which no further significant improvement in treatment efficiency may be expected. In fact, any increase in size above this volume appears to have no measurable effect except to increase the rate of compressed air required. For the Owls Head plant this critical aeration period is one hour, equivalent to an aerator loading of 200 lbs. of raw sewage B.O.D. per day per 1000 cu. ft. of volume.

Applications of Short Period Aeration

A region of operation for the modified aeration process from zero detention time to the minimum necessary for full treatment has been defined. This region of operation, which has received little application, comprises such low aeration periods as to justify calling it "short period" aeration.

Short period aeration can be advantageously applied to full scale modified aeration process plants. By using multiple aeration units, with high hydraulic inlet and outlet capacities and a flexible air diffusion system, the plant may be operated at selected points along the short period aeration curve. At such times as maximum B.O.D. removal efficiency is required, enough aeration units are placed in service to attain the top portion of the curve. On the other hand, when seasonal demands for treatment efficiency are lower, aeration units are removed from service to maintain the required removal and, at the same time, to realize significant savings in air requirements. Of course, the sludge age must be adjusted for optimum efficiencies under these various conditions.

Plain sedimentation plants are inherently adaptable to the use of short period aeration because the sedimentation tanks are readily convertible for final settling. The two or more hours of detention usually available will provide suitable overflow rates for separation of the aerator effluent solids. Aeration tanks, loaded at rates approximating 200 lbs. per day of raw sewage B.O.D. per 1000 cu. ft., and with area minimized by allowable tank depths of 15 feet, can often be accommodated on existing plant sites. Plant buildings will usually be adequate to house the air compression equipment. The "new" final settling tanks must be equipped with non-clogging pumps for returning sludge to the aeration tanks. Inasmuch as the small aeration tanks will require fairly high mixed liquor suspended solids concentrations to attain the requisite sludge age, the ratio of return sludge to excess sludge must be kept high. If final settling tanks are to be used for withdrawing solids from the process, a minimum of five units must be available. If not, a suitable separate sludge thickening system should be provided. In return for these comparatively small expenditures for plant equipment and structure, a short period aeration system can be installed to significantly improve the B.O.D. removal efficiency of the plant.

An especially attractive application of short period aeration is in those plain sedimentation plants which are already equipped with preaeration facilities of about one-half hour detention. By including return sludge pumps, augmenting air diffusion facilities and providing some sludge thickening system, the B.O.D. removal efficiency of the plant can be markedly improved.

This may well answer the need of a locality which is required by local stream conditions to raise its standards of treatment without exceeding site and budget allowances. Thus, short period aeration can be used in some instances to attain satisfactory sanitary stream conditions while not requiring the high expenditures for complete treatment demanded by the conventional or high-rate trickling filters or the activated sludge process.

ACKNOWLEDGMENT

The authors wish to acknowledge with appreciation the assistance of the following associates in solving the many problems presented during this extended plant scale investigation: T. Margolies, Plant Chemist; F.H.A. Lino, Plant Superintendent; A.H. Chasick, Engineer of Treatment; and A.J. Scanlon, Chief of the Bureau of Sewage Disposal Operation.

The authors are especially grateful for the advice and guidance of W.A. O'Leary, Director of the Division of Sewage Disposal.

REFERENCES

1. Setter, Lloyd R. "Modified Sewage Aeration, Part I," Sewage Works Journal, 15, 4, 629 (July 1943).

2. Setter, L. R. and Edwards, G. P. "Modified Sewage Aeration, Part II," Sewage Works Journal, 16, 2, 278 (March 1944).

3. Steffensen, S. W. "New York City's Owls Head Pollution Control Project," Sewage and Industrial Wastes, 24, 4, 445 (April 1952).

4. Torpey, W. N. and Chasick, A. H. "Principles of Activated Sludge Operation," Sewage and Industrial Wastes, 27, 11, 1217 (November 1955).

Table I

ANNUAL RESULTS OF TREATMENT

Year	Sewage Flow M.G.D.	Return Sludge %	Air Ratio cu ft /gal	Aer. Period Hrs.	Over-Flow Rate gals/ sq ft /day	Sludge Age Days	Aer. Effl. S.S. ppm	Sludge Density Index	Suspended Solids Raw ppm	Suspended Solids Final ppm	Suspended Solids Removal %	B. O. D. Raw ppm	B. O. D. Final ppm	B. O. D. Removal %	Filtrate B.O.D. Raw ppm	Filtrate B.O.D. Final ppm
1952	81.0	11	.51	2.4	790	.25	400	-	167	45	73	172	68	60	73	38
1953	84.7	15	.47	1.9	780	.21	410	2.3	169	48	72	165	70	58	73	41
1954	83.4	13	.48	2.1	770	.21	370	1.7	172	44	74	160	59	63	67	36
1955	82.9	9	.42	2.3	770	.18	280	2.2	160	45	72	167	70	58	70	41
Aver.	83.0	12	.47	2.2	780	.21	360	2.1	167	46	73	166	67	60	71	39

Table II

RESULTS OF TREATMENT BY MODIFIED AERATION
AT LOW AERATION PERIOD

Month	Sewage Flow	Ret. Sludge	Air Ratio	Aer. Period	Over-Flow Rate	Sludge Age	Aer. Effl. S.S.	Sludge Density Index	Suspended Solids			B. O. D.			Filtrate BOD		Sewage Temp.
	M.G.D.	%	cu ft /gal	Hrs.	gals/ sq ft /day	Days	ppm		Raw ppm	Final ppm	Removal %	Raw ppm	Final ppm	Removal %	Raw ppm	Final ppm	°F.
March	90.8	17	.21	.53	830	.22	980	2.8	141	50	65	163	77	53	77	45	57
April	87.6	19	.22	.55	800	.21	1010	2.6	162	50	69	158	77	51	70	46	59
May	82.7	22	.26	.56	660	.28	1360	2.8	173	44	75	191	84	56	85	53	62
June	87.9	21	.26	.53	710	.26	1300	2.8	176	47	73	162	66	59	62	41	70
July	87.2	21	.18	.53	780	.23	910	2.7	133	43	68	141	60	57	53	32	71
Aug.	88.5	20	.22	.53	810	.19	790	2.2	144	47	67	119	51	57	45	26	73
Sept.	87.3	19	.22	.54	800	.12	590	2.2	164	57	65	150	76	49	65	43	71
Oct.	86.7	19	.24	.55	800	.16	740	1.9	158	61	61	171	83	51	73	51	68
Nov.	85.3	19	.26	.55	780	.16	740	2.4	159	56	65	181	92	49	87	65	63
Dec.	87.6	19	.25	.54	800	.19	820	2.5	150	47	69	166	80	52	69	49	61
Jan.	75.0*	16	.27	.62	700	.24	1030	3.0	157	46	71	177	75	58	73	48	55
Feb.	76.8*	21	.24	.61	710	.21	820	2.1	147	45	69	140	74	47	65	43	57
Aver.	85.4	19	.24	.55	770	.20	920	2.5	155	49	68	160	75	53	69	45	-

*Some flow decrease during major alteration of sewage system.

Table III

RESULTS OF TREATMENT AT VARIOUS AERATION PERIODS

No. of Aerators	Full Months of Operation	Aer. Period	Loading Rate	Flow	Ret. Sludge	Air Ratio	Over-Flow Rate	Sludge Age	Aer. Effl. S.S.	Suspended Solids			B. O. D.			Filtrate B.O.D.	
										Raw	Final	Removal	Raw	Final	Removal	Raw	Final
		Hrs.	Lbs BOD per day per 1000 cu ft of aerator	MGD	%	cu ft /gal	gals/ sq ft /day	Days	ppm	ppm	ppm	%	ppm	ppm	%	ppm	ppm
4 + Channel	8	2.7	90	85	11	.51	790	.24	310	157	41	74	175	75	57	78	45
3 + Channel	26	2.2	100	83	11	.46	770	.21	340	167	45	73	164	64	61	67	38
2 + Channel	5	1.7	120	82	15	.47	740	.18	390	180	51	72	159	64	60	69	37
1 + Channel	2	1.1	180	85	18	.43	790	.19	590	178	52	71	164	61	63	70	35
0 - Channel	12	0.55	370	85	19	.24	770	.20	920	155	49	68	160	75	53	69	45

Table IV

COMPARISON OF EFFLUENT QUALITY BELOW ONE HOUR AERATION

Grab vs. Automatic Sampling

Month	FINAL EFFLUENT BY GRAB SAMPLES Suspended Solids ppm				Final Effl. by Auto. Sampler	FINAL EFFLUENT BY GRAB SAMPLES B.O.D. ppm				Final Effl. by Auto. Sampler	FINAL EFFLUENT BY GRAB SAMPLES Filtrate B.O.D. ppm				Final Effl. by Auto. Sampler
	20 min. Aer.	35 min. Aer.	60 min. Aer.	Aver.	S.S. ppm	20 min. Aer.	35 min. Aer.	60 min. Aer.	Aver.	B.O.D. ppm	20 min. Aer.	35 min. Aer.	60 min. Aer.	Aver.	Filtrate B.O.D. ppm
May	46	44	29	43	44	91	87	71	83	84	58	59	49	55	53
June	44	42	40	42	47	77	70	63	70	66	49	46	42	46	41
July	41	40	38	40	43	63	64	60	62	60	42	41	39	41	32
August	42	40	40	41	47	53	51	45	50	51	32	33	27	31	26
Sept.	59	55	53	56	57	76	71	67	71	76	45	45	44	45	43
Oct.	52	54	52	53	61	87	87	74	80	83	55	51	46	51	51
Nov.	56	54	51	54	56	104	99	85	96	92	77	75	65	72	65
Dec.	51	44	44	46	47	83	77	70	77	80	63	57	50	57	49
Jan.	49	45	43	46	46	82	78	71	77	75	49	51	45	48	48
Feb.	48	51	43	47	45	81	80	71	77	74	46	45	42	44	43
Aver.	49	47	43	46	49	80	76	68	75	74	52	50	45	49	45

Figure 1

DIAGRAMMATIC LAYOUT OF AERATION AND SETTLING TANKS

Owls Head Pollution Control Project

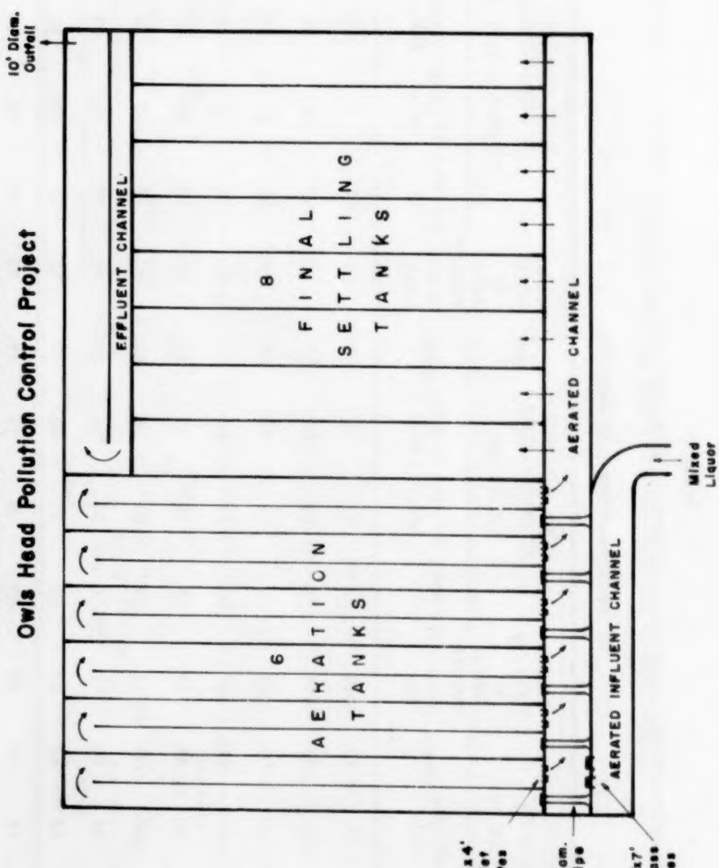
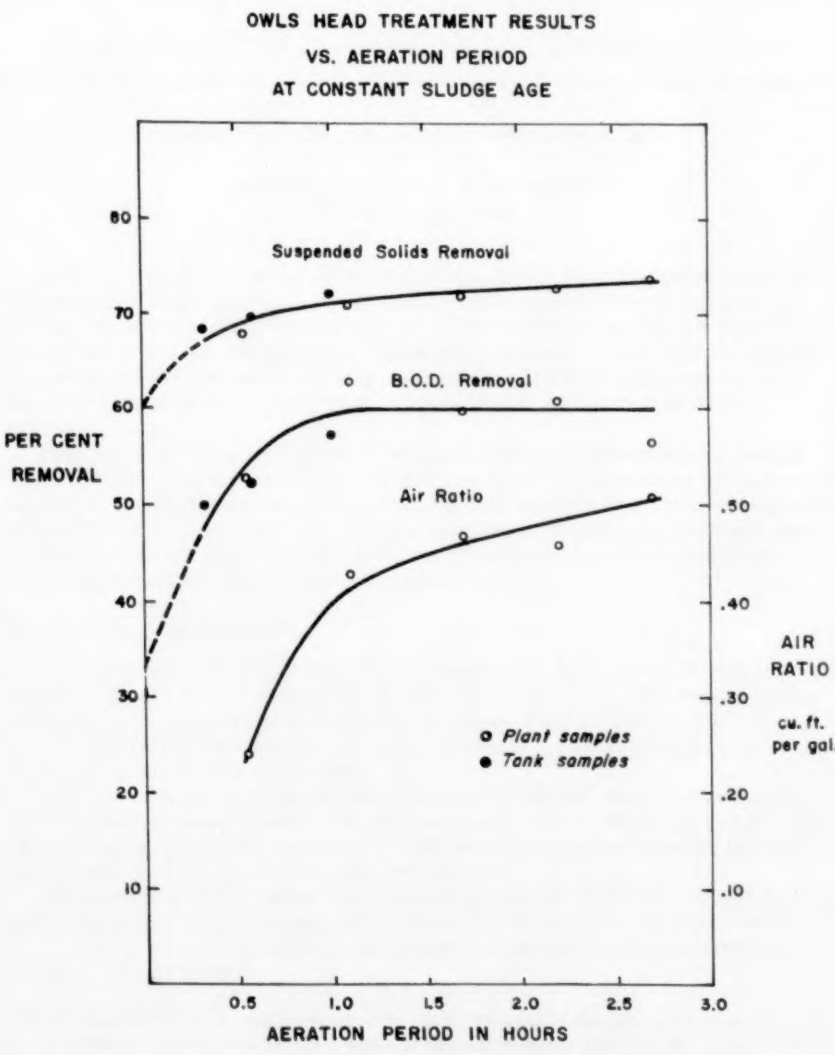


Figure 2


OWLS HEAD TREATMENT RESULTS
VS. AERATION PERIOD
AT CONSTANT SLUDGE AGE
Suspended Solids Removal
B.O.D. Removal
Air Ratio
PER CENT
REMOVAL
AIR
RATIO
cu. ft.
per gal.
Plant samples
Tank samples
AERATION PERIOD IN HOURS

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN OF WATER SUPPLY STRUCTURES[a]

Howard J. Carlock,[1] A. M. ASCE
(Proc. Paper 1682)

Problems in design arise on most projects which require modification of a standard design or development of a new idea to satisfy site or location conditions. In many instances the engineer feels that the ideas he has developed are not of sufficient magnitude or importance to write a paper for the Society journals, and the pressure of business has made it difficult for some engineers to take time to prepare a paper on a subject of interest to Society members.

This paper is a resumé of seven subjects related to water supply design which may be of some interest. The designs to be discussed are not presented as new developments in engineering, but are designs that have, modified in some instances, been built and put into operation. No attempt has been made to review the literature, and the specific references made are companies owned by American Water Works Company, Inc.

Multiple Purpose Reservoirs

In 1949-1950 The Alexandria Water Company of Alexandria, Virginia, constructed a new surface supply works and treatment plant. The supply works consisted of a concrete gravity-type dam impounding 57 million gallons, a raw water intake and a low service pumping station to pump raw water to a treatment plant a half-mile away.

The low service pumping station contains a 350 KW. water turbine generator to provide standby power for operation of some of the pumps; and in addition, two pumps each requiring 200 HP were provided with water turbine drives and electric motors as auxiliary drives.

Most of the year the water turbines are operated to provide electric power and to drive the 200 HP pump units. During periods when the water in the reservoir is low, the pumps are operated from power purchased from the local power company.

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1682 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

a. Presented at meeting of ASCE, New York, N. Y., October, 1957.

1. Chief Design Engr., American Water Works Service Company, Inc., Philadelphia, Pa.

The rapid increase in population and demand on the water supply system since 1950 necessitated the construction of the second stage in the development of the supply works.

The second stage construction was started in 1956 and is now being completed. It consists of a second dam and reservoir, constructed 3000 feet upstream from Reservoir No. 1. Dam No. 2 is a concrete gravity-type structure 65 feet high and impounds 10 billion gallons of water. The base of Dam No. 2 is slightly higher than the top elevation of Dam No. 1. A power house was built as part of the dam and intake and houses two 500 KW. water turbine-driven generators.

The proposed method of operation consists of discharging the water from Reservoir No. 2 through the two 500 KW. generators into Reservoir No. 1. The water is then passed through the 350 KW. generator and the two turbines operating the 200 HP pumps at the Low Service Station at Reservoir No. 1. This operation provides sufficient power for the low service pumps and most of the power requirements for the high service pumps at the treatment plant. The maximum amount of purchased power required for this operation during peak demands will be less than 700 KW. at the present time.

The low service pumps are piped so that they can take water from either reservoir. The normal operating procedure is to take water from Reservoir No. 2 to utilize the head available and increase the capacity of the low service pumps.

It is evident that with this arrangement of the two reservoirs there are many possible ways of operating the pumps and generators to take full advantage of the water available at any season of the year.

Use of Electric Current for Prevention of Ice Formation at Intake Structures

One of the more serious problems in cold climates is the formation of ice at the intake works. One means of alleviating this condition is by use of a heating cable. A heating cable is a series of high resistance wires in a lead covering. When a current is passed through the wires, the lead covering gets warm and transmits heat in its immediate area. To be of benefit, the cable should be in contact with a heat conductor, such as metal. The length of cable and wattage applied determine the temperature differential that can be maintained. The heating cable principle has been used at the Lake Erie intake of The Ashtabula Water Works Company of Ashtabula, Ohio, and for reservoir intakes at the Commonwealth Water Company, Summit, New Jersey.

At Ashtabula, a 30-inch concrete pipeline was extended into Lake Erie to serve as a pipe intake. A crib was constructed on the inlet end of the pipeline with a metal grating on top. A power line was laid in the trench with the pipeline and a heating cable installed on top of the grating. Sufficient current is applied during the winter months to prevent the formation of frazzle and anchor ice which are common in the Great Lakes area. Frazzle ice usually forms a mat and gradually builds up sufficiently to clog intake structures.

The Commonwealth Water Company installed vertical turbine-type pumps in an open reservoir to pump raw water to the treatment plant. The pumps are installed outdoors on a steel framework with the suction columns exposed to the weather and still water in the reservoir, subject to freezing. Heating cables are wound around each suction column to heat the columns sufficiently to prevent ice formation at the water surface when the pumps are not operating.

Heating cables can be fastened to bar racks, valves, sluice gates, screens and other equipment exposed to the weather and in contact with water to prevent damage from ice.

Well Supply Recharge Basins

The increasing demands on water supply systems and periods of prolonged droughts have resulted in lowering the ground water table at many plants using wells as a source of supply.

The Peoria Water Works Company of Peoria, Illinois, had such a problem and constructed a recharge basin for the purpose of putting water back into the ground to raise the water table. Its main plant is located on the Illinois River and obtains its supply from shallow wells varying in depth from 40 to 60 feet.

A natural gravel bed was uncovered adjacent to the plant and earth dikes constructed to create a basin having an area of 1.85 acres and providing a water depth of ten feet. Water from the Illinois River is pumped into this basin at a rate of 5 million gallons per day, or 2.7 million gallons per acre per day. A six-inch layer of sand was spread on top of the gravel to filter out any sediment in the water and to prevent clogging of the gravel. Water is pumped into the basin in the wintertime when it is cold so that it will maintain as low a temperature of the ground water as possible. The water is chlorinated as it goes into the basin (at a rate of 6 parts per million) to satisfy the chlorine demand of the water.

The pump is operated approximately eight months per year, and the basin is cleaned during the summer. In this particular instance the water in the river does not contain much suspended material since the level is controlled by a dam located a few miles downstream, and this dam has created a lake in the vicinity of the plant which acts like a settling basin.

Side Stream Storage

The engineer is faced with many problems in designing an impounded supply. In many instances the topography or foundation conditions are such that the cost of constructing a dam is not economical for the amount of storage to be obtained. In addition, the requirements for maintaining minimum flows for riparian rights and providing adequate spillway capacity for maximum flood flows require considerable expenditures.

A method of providing storage where conditions are not suitable for construction of a dam across the stream is the side stream storage reservoir. This type of reservoir is located adjacent to or in close proximity to the stream and is usually constructed with earth embankment walls. Material is excavated on the site to provide the embankment enclosure. Concrete walls could be used and possibly steel sheet piling, depending on conditions and cost. The inside of the reservoir should have an impervious blanket such as clay to minimize leakage.

This type of reservoir does not require expensive spillway or overflow structures since it is not affected by stream flows and does not obstruct the waterway. There are a number of ways of filling this type reservoir, two such means are by pumping and by gravity flow.

To fill by pumping requires the installation of pumps at the stream that pump directly to the reservoir. When there is sufficient slope to the stream, it may be possible to fill the reservoir by gravity. This can be accomplished

by constructing a low dam upstream, usually two to five feet high and connecting a pipeline from the dam to the reservoir located at a lower elevation. Where this system is used it is necessary to provide an overflow in the storage reservoir with capacity equal to the capacity of the fill line to control the maximum level in the reservoir.

The Commonwealth Water Company at Summit, New Jersey, employs such a system for raw water storage. Water is pumped from the Passaic River during periods of adequate flow in the river and stored in three reservoirs having a total capacity of two billion gallons. The three reservoirs are at different elevations, and means are provided for discharging the water from the highest reservoir to the other two reservoirs by gravity.

This same system was used by the Kokomo Water Works Company at Kokomo, Indiana, for a reservoir storing 260 million gallons and by The Maryland Water Works Company, Bel Air, Maryland, for a reservoir having a capacity of 120 million gallons.

The White Deer Mountain Water Company of Milton, Pennsylvania, constructed a side stream reservoir having a capacity of 400 million gallons. This reservoir is filled by gravity from a pipeline connected to a low dam upstream.

Filling of this type of reservoir usually takes place in the period from October to May when there is ample flow in the streams. It is not necessary to maintain minimum flow in the stream from the reservoir storage because the storage is obtained during periods of maximum discharge in the spring and fall, and during the summer months the pumps used for filling are not operated.

Access Railway

The Lexington Water Company is constructing a new water supply works and treatment plant in Lexington, Kentucky. The treatment plant is located at the top of a bluff, and the intake works and low service pumping facilities are located on the river 350 feet below. Access to the intake works is by boat or from the top of the bluff. Since it is necessary for the plant operators to have an easy means of access between the plant and the intake works, an inclined railway was designed to travel up and down the bluff at an angle of approximately 42° for a distance of 510 feet. The railway is designed similar to a scenic railway found at amusement parks. An open car having a capacity of 2000 pounds rides up and down the rails and is operated by a push button mounted on the car. Additional button stations are installed at the top and bottom for calling the car. The railway was designed with the assistance of a manufacturer of scenic railway equipment and contains all of the safety features incorporated in scenic railway design. The car travels at a speed of 100 feet per minute which is approximately walking speed. The application is unusual and required considerable investigation to find a firm willing to construct a car for this purpose with all of the safety features necessary for one-man operation.

Spillways

In the design of dams, one of the problems is the provision of adequate spillway capacity. In many states the maximum flood discharge is established by state agencies having jurisdiction over the rivers and streams. The curves and formulae they recommend provide for the maximum flood anticipated for

the streams and are usually far in excess of the maximum flood on record for a particular stream.

In many instances the engineer may select a site for a dam that is most desirable from the construction cost standpoint; but when he gets to the design of the spillway, he finds that even by using the full length of the dam as a spillway, there is not sufficient length to provide the necessary capacity within the head limitations.

In such an instance the use of a "Fuse Plug" may be of some value. A Fuse Plug is usually an earth fill section of dam that is designed to wash out when the reservoir level reaches the design capacity of the normal spillway. A Fuse Plug is usually constructed at one end of a dam, sometimes even at a remote location, and consists of a channel cut in the side hill of sufficient depth and width to carry the required flood flow beyond the toe of the dam. An earth dam is constructed across the channel with the top of the dam at the elevation of the design maximum water level of the spillway. When the spillway reaches its design capacity, the Fuse Plug is topped and washes out to a depth and width necessary to discharge the excess flow.

When such a flood occurs, there is a cost of replacing the earth fill section and perhaps repairing downstream channel, but the initial cost of construction of a Fuse Plug is considerably less than the equivalent concrete spillway and appurtenant structures which may never be called upon to function at its maximum.

In earth fill dam construction, when a Fuse Plug is used it is necessary to protect the end of the dam (with a concrete wall) and sometimes to construct a cut-off wall under the Fuse Plug to limit the area designed to wash out.

An economical type of spillway that has been used in dam construction is the morning glory spillway. This type of spillway consists of a vertical shaft on the upstream side of the dam with a flared open top section at maximum water line. The flared section acts as a circular weir. The vertical section is connected to a conduit extending through the dam at its base. A stilling basin is usually provided at the outlet end of the conduit to dissipate the energy of the flood flow.

A morning glory spillway and Fuse Plug were used with an earth dam and reservoir for the Greenwich Water Company, Greenwich, Connecticut. A concrete gravity-type spillway and Fuse Plug were used with an earth dam and reservoir for the Kokomo Water Works Company at Kokomo, Indiana. The Municipal Authority of Westmoreland County, Westmoreland, Pennsylvania, uses the morning glory spillway with an earth dam with provision for bypassing maximum floods.

Butterfly Valves

Butterfly valves of the rubber seated tight-closing type are being used more often in intake structures and outlet works at dams. In addition to being tight-closing, they have the advantage of ease of operation and an accuracy range of 25 to 1 for control. They have been installed in intakes and in pipelines extending through a dam to control downstream flow for riparian owners and maintain minimum flow. These valves can be controlled manually, hydraulically, pneumatically, or electrically, and can be controlled by any primary measuring device such as an orifice plate, Parshall flume, venturi tube or weir.

Butterfly valves have been used on raw water supply lines to treatment works to control the flow. They are usually controlled by the water level in the settling basins and will automatically throttle the raw water flow whether gravity or pumped as the water rises in the basins when filter capacity is reduced and prevent the settling basins from overflowing.

CONCLUSION

These are but a few of the many ideas and applications that engineers are continually developing in their work; and with the continuing increase of construction costs, further improvements in design will be made to provide the same facilities at minimum cost.

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SALARIES OF LOCAL ENVIRONMENTAL HEALTH PERSONNEL IN 1956

Report of the Committee on Salaries
Conference of Municipal Public Health Engineers
(Proc. Paper 1685)

SECTIONS II, III and IV[a]

This is the continuation of a Report prepared as a result of a survey of some 3,251 professional positions in environmental sanitation divisions in 371 local health departments. A detailed discussion of how the study was conducted and general data covering all groups of positions was presented in Section I of the Report and will not be repeated here.

SECTION II - Engineers

A. All Engineers

A total of 157 filled engineering positions were included in the study. Their median maximum salary in 1956 was $7,375.*

1. Educational Requirements

Of the 157 positions, 146 had a regular basic educational requirement of a bachelor's degree or higher. Of the remaining 11, 2 required two years of college; 8 required high school graduation; and 1 accepted less than a high school graduation. Fifty positions, nearly one-third, required a master's degree for appointment. Chart IV shows a direct relationship between educational requirements of engineering positions and salary offered. Most of these positions did not permit substitution of experience for education below the bachelor level.

Of the 96 positions requiring a bachelor's degree, 82 required a bachelor's degree in engineering, 9 required a bachelor in sanitary science or public health, 2 accepted any bachelor in science, and 1 any bachelor's degree. (No information was provided for the remaining two).

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1685 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

a. Section I appeared as Proc. Paper 1282, June 1957.
*Not $7,140 as reported in error in SECTION I, Proc. Paper 1282.

Of the 50 positions requiring a master's degree, 11 required a master in public health, 25 a master in sanitary engineering or public health engineering, 6 a master in public health, sanitary engineering or public health engineering, 1 a master of sanitary science, 1 any master of science, and 1 a master in public health or any master in engineering. (No information was provided for the remaining five.) It is interesting to note that the 11 positions requiring a master of public health offered a median maximum salary of $9,750 as compared with $8,250 for the 25 positions requiring a master in sanitary engineering or public health engineering.

2. Experience Requirements

a. Total Experience

Only 18 of the 157 positions were available to engineers with no experience. Experience requirements were distributed among the 157 positions as follows:

TABLE 9

Years Experience Requ'd.	0	1	2	3	4	5	6	7	8	10 or more	No Info.	Total
Number of Positions	18	10	19	15	16	37	10	3	7	11	11	157

Chart V shows the relationship between total experience required and median maximum salary. There appears to be little change in salary level in the "0-2" and "5-8" year experience levels, with most of the rise found in the "2-5" and "10 years and more" levels.

b. Administrative or Supervisory Experience

Half of the 157 engineering positions required no supervisory or administrative experience, the balance required between 1 and 10 or more years of such experience. There seems to be an approximate relationship between the years of such experience required and the median maximum salary offered. This is shown in Chart VI.

3. Registration Requirements

Half of the engineering positions in the study require professional registration or eligibility for such registration. Positions having such a requirement pay a median maximum salary of $8,160 as compared with $6,570 for those which do not have this requirement.

4. Supervisory Responsibility

Of the 157 engineers under study, 138 had some supervisory responsibility over professional personnal. Chart VII shows that there is a relationship between the median salary paid to engineers and the number of professional personnel they supervise. The relationship is not consistent and it appears that engineers supervising from 40 to 75 persons are not compensated sufficiently for their supervisory responsibility.

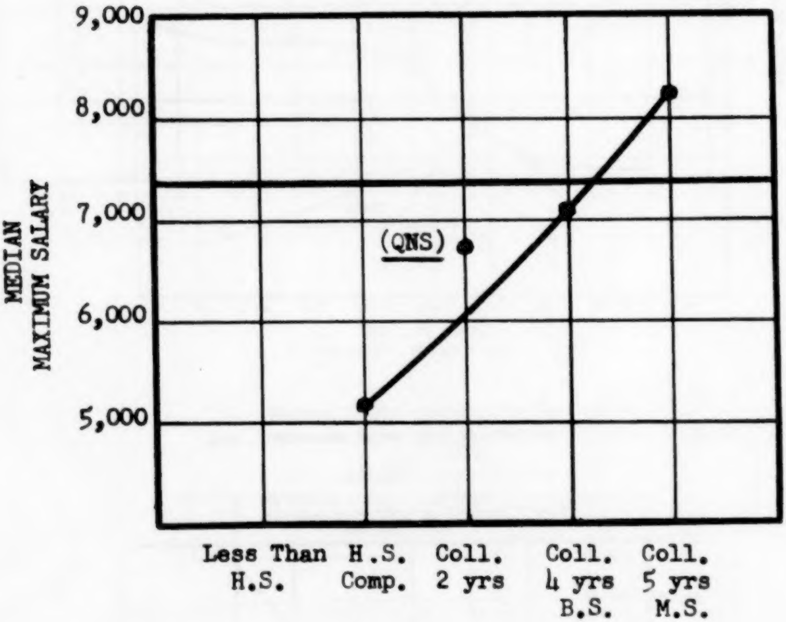
5. Population Served

Chart VIII shows that the salaries of engineering positions in local health departments bear a direct relationship with the size of the population group

ENGINEERS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART IV

EDUCATIONAL LEVEL REQUIRED FOR APPOINTMENT



EDUCATIONAL LEVEL

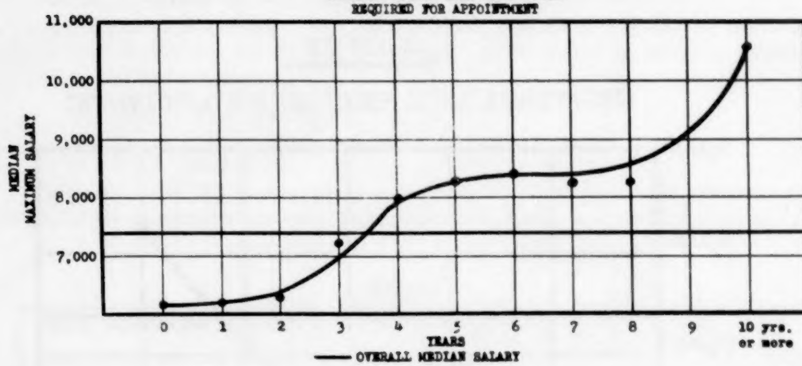
—— OVERALL MEDIAN SALARY

QNS - QUANTITY NOT SUFFICIENT TO PROVIDE SIGNIFICANT INFORMATION

ENGINEERS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART V

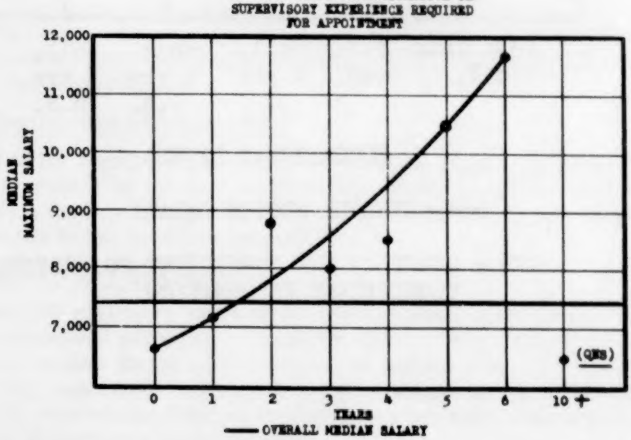
MINIMUM TOTAL YEARS EXPERIENCE
REQUIRED FOR APPOINTMENT



ENGINEERS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART VI

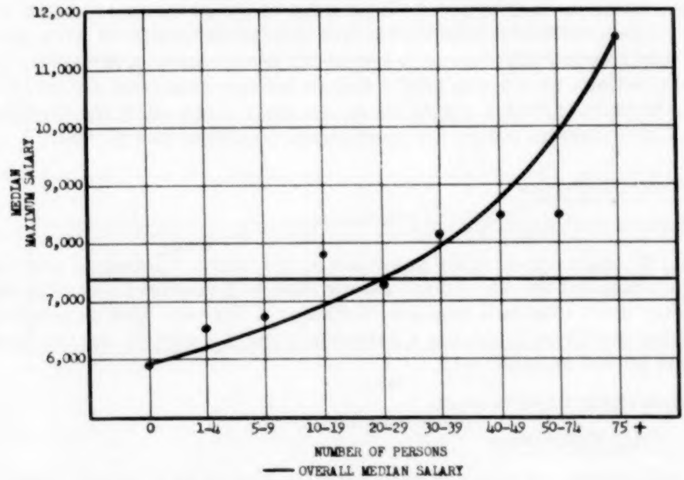
MINIMUM YEARS ADMINISTRATIVE OR
SUPERVISORY EXPERIENCE REQUIRED
FOR APPOINTMENT



ENGINEERS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART VII

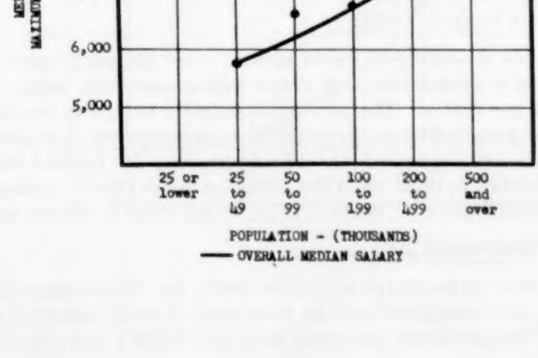
NUMBER OF PROFESSIONAL PERSONNEL SUPERVISED



ENGINEERS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART VIII

POPULATION SERVED BY POSITION



for which they are responsible. "Population Served" is only one of a number of measures of the amount of responsibility of a position, others are: degree of specialization and the total number of program activities supervised.

6. Generalization vs. Specialization

Two-thirds (107) of the 157 engineering positions were reported as generalized insofar as they covered all or most of the programs handled by a division of environmental sanitation. The remaining positions were specialized as they handled only one or a few of the programs of a division. Generalized positions as a group paid a higher median maximum salary ($7,660), probably because included among them are most if not all of the directors. The median maximum salary for specialized positions was $7,160.

B. Engineer Directors

1. Educational Requirements

Among 67 engineer director positions in the study, 1 admitted persons with less than a high school education. Fewer than half required a college degree, and slightly more than half required a master's degree. Median maximum salaries for positions requiring a bachelor's and a master's degree were $8,056 and $8,395 respectively.

2. Experience Requirements

a. Total Experience

The total number of years of experience seems to be a factor of only slight importance in determining salary for engineer directors. There is some increase between "0" and "4" years' experience and again between "7" and "10" years. This is shown in Chart IX.

b. Administrative or Supervisory Experience

It is interesting to note that 5 of the 67 engineer director positions required no experience at all. Twenty-three engineer director positions require no supervisory or administrative experience. Salaries of engineer director positions requiring more than 4 years administrative or supervisory experience are considerably higher than those requiring less. This is shown in Chart X. There seems to be little salary credit given to positions requiring less than 4 years of such experience.

3. Supervisory Responsibility

Among engineer directors there is less of a relationship between salary and the number of professional employees supervised than among local public health engineers generally. The median maximum salary of engineer directors increases in proportion to the number of professional employees supervised where there are between 1 and 19 employees. No further increase in salary levels appears evident until the division supervised reaches a size of 75 professional employees or more. This information is shown in Chart XI.

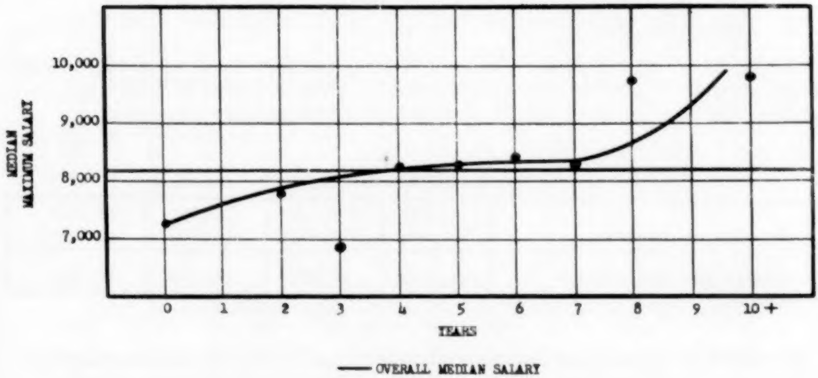
4. Program Responsibility

The total number of program activities (such as: Water Supply Sanitation, Food Sanitation, etc.) administered by directors of environmental sanitation in each of the 35 departments referred to in SECTION I was compared with

ENGINEER DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART IX

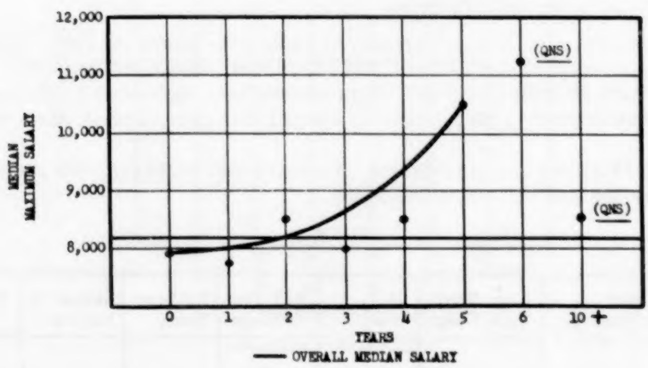
MINIMUM TOTAL YEARS EXPERIENCE REQUIRED FOR APPOINTMENT



ENGINEER DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART X

MINIMUM TOTAL YEARS ADMINISTRATIVE OR SUPERVISORY EXPERIENCE REQUIRED FOR APPOINTMENT



director salaries. This comparison, even when director positions were grouped by population served, did not disclose any relationship between the number of activities administered and median maximum salary. For example, a director for a county serving a population of 130,000 with 7 such activities was paid a higher salary than a director for a city of 750,000 population with 11 activities. While this is an extreme example, no pattern appears to relate the number of activities with the director's salary, even when population is considered.

5. Population Served

As could be expected, the salaries of engineer directors vary with the size of the population served by the department. This is shown in Chart XII. Median maximum salary levels of engineer directors were as follows:

TABLE 10

Population Served	50,000 99,000	100,000 199,000	200,000 499,000	500,000 and over
Median Maximum Salary	$6,850	$8,000	$8,050	$9,940

It should be noted here that in each population class, the salary range is quite broad. For example, in the 100,000 - 199,000 population group the lowest salary is $5,250 and the highest $9,250. The same is true where director salaries are grouped by the number of persons supervised.

SECTION III - Sanitarians

A. All Sanitarians

A total of 2,893 filled sanitarian positions were included in the study. Their median maximum salary in 1956 was $4,776.

1. Educational Requirements

As the educational requirements of sanitarian positions rise, there is a slight improvement in salaries. This is clearly shown in Chart XIII. Comparing Chart IV with Chart XIII, it is evident that engineering salaries rise much more directly with increasing educational requirements than sanitarian salaries.

Table 11 shows the distribution of educational requirements and salary levels for the sanitarians in the study.

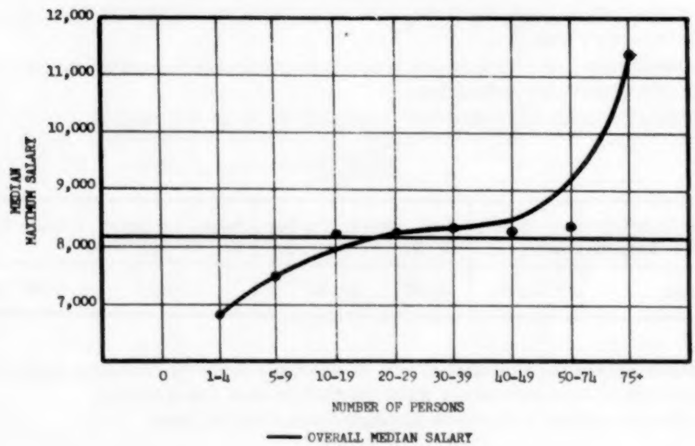
TABLE 11

Min. Educ. Level Requ'd.	Less Than H.S. Comp.	H.S. Comp.	1-3 Yrs. College	College Grad.	Master's Degree	Total
# Sanitarians	146	877	453	1357	51	2884
% Sanitarians	5.0%	30.5%	15.7%	47.0%	1.8%	100.0%
Med. Max. Sal.	$4,250	$4,315	$4,814	$5,220	$5,965	$4,776

ENGINEER DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XI

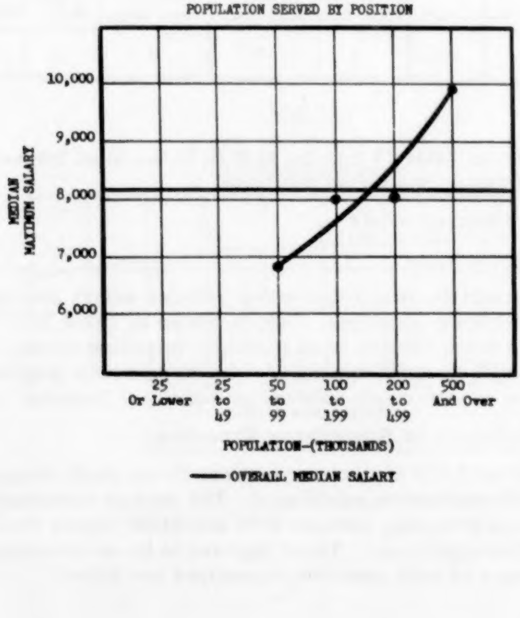
NUMBER OF PROFESSIONAL PERSONNEL SUPERVISED



ENGINEER DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XII

POPULATION SERVED BY POSITION



Nearly 50% of all sanitarian positions at the local level now require a college degree; an additional 16% require some college education. This is shown in Table 11. Jobs requiring college graduation pay approximately $900.00 more than jobs requiring high school graduation. Positions requiring a master's degree pay about $750.00 more than those requiring a bachelor's degree. The study disclosed an extremely small percentage of sanitarian positions requiring a master's degree. Sixteen of the 51 positions requiring a master's were directors.

Approximately one-third of all sanitarian positions permitted some substitution of experience for education.

The types of bachelor's degrees required were as follows:

TABLE 12

Type Bachelor Degree Requ'd.	Any Bach. Degree	Any B. S.	Bach. in San. Sc. or P.H.	Bach. in Dairy Sc. or An. Hsb.	Other	Total
Percent	12.4%	41.6%	28.6%	11.4%	6.0%	100%

Positions requiring the bachelor's in sanitary science or public health offered salaries of approximately $300.00 higher than the average.

The type of master's degrees required were as follows:

TABLE 13

Typ. Master's Degree Requ'd.	M.P.H.	M.S. in San. Sc.	M.P.H. or M.S. in San. Sc.	M.P.H. or M.S. in San. Eng.	Other M.S.	Other	Total
	22	3	9	9	2	6	51

It is evident from Table 13 that the M.P.H. is the most frequently required master's degree among sanitarian positions.

2. Experience Requirements

a. Total Experience

There is surprisingly little relationship between salary and experience requirements of sanitarian positions. This is shown in Table XIV. There is a slight increase of some $300.00 from positions requiring no experience to positions requiring 3 years of experience, but beyond this, positions with higher experience requirements hold out no additional financial reward.

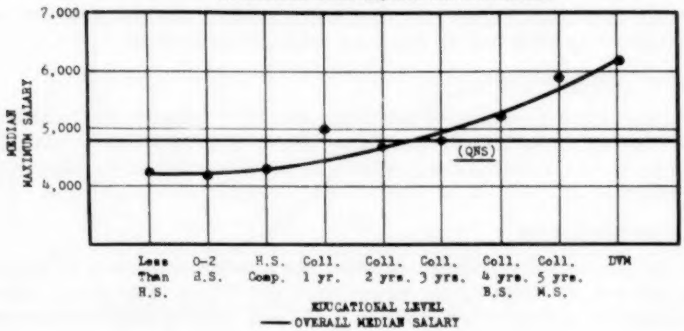
b. Administrative or Supervisory Experience

Six percent of the 2,893 sanitarian positions in the study required some supervisory or administrative experience. The median maximum salary of these positions was generally between $500 and $1000 higher than for sanitarians without this experience. There appears to be no relationship between the number of years of such experience required and salary.

SANITARIANS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XIII

EDUCATIONAL LEVEL REQUIRED FOR APPOINTMENT

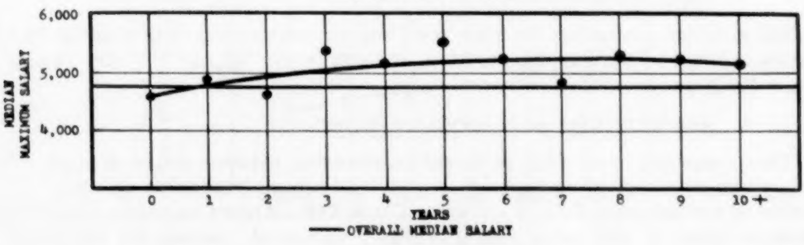


SANITARIANS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XIV

MINIMUM TOTAL YEARS EXPERIENCE

REQUIRED FOR APPOINTMENT



3. Registration Requirements

The salary levels of sanitarian positions which require registration or eligibility therefor are higher than those which do not have that requirement. This is probably due in part to the fact that these requirements do not apply to entrance positions and that they encourage higher educational requirements. Sixteen percent of the sanitarian positions in the study required registration or eligibility therefor. These positions had a median maximum salary level of $990.00 above those which do not have such a requirement.

4. Supervisory Responsibility

The salary level of sanitarian positions clearly increases as the number of professional personnel supervised increases. This is shown in Chart XV. Twenty percent of all sanitarian positions in the study had some supervisory responsibility.

5. Population Served

There is rather little variation between the population served by sanitarian positions and salary. This is shown in Chart XVI. This is in sharp contrast to the engineering group where there is a distinct relationship between these two factors.

6. Generalization vs. Specialization

Three-fifth of the sanitarian positions in the study were reported to be generalized or covering all or most of the programs handled by the division of environmental sanitation. The salary levels were slightly and perhaps insignificantly lower for the generalists than for the specialists.

B. Sanitarian Directors

1. Educational Requirements

Sanitarian director positions requiring 2 years of college education or more offer higher salaries as educational requirements increase. This is shown in Chart XVII.

2. Experience Requirements

a. Total Experience

Salaries for sanitarian director positions increase somewhat with the experience requirements of the position, at least in the "0" to "5" years range. Chart XVIII presents this data.

b. Administrative or Supervisory Experience

There appears to be a fairly direct relationship between years of supervisory and administrative experience required among sanitarian director positions and salary. This is shown in Chart XIX. Among engineer directors, shown in Chart X, this relationship is less pronounced. Among the 196 sanitarian director positions in the study, there were only 6 which required more than 3 years of administrative or supervisory experience. More than half of these positions (123) required no such experience at all.

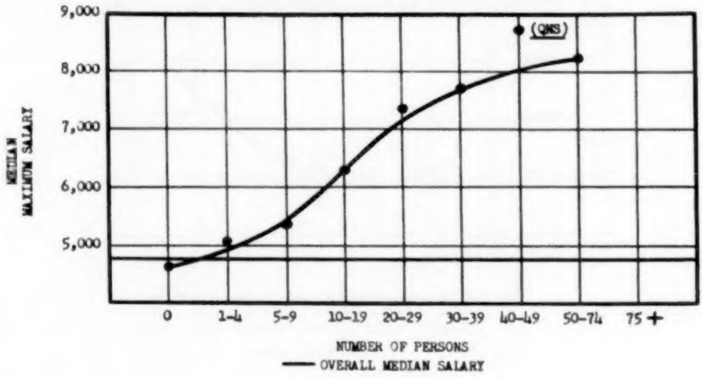
3. Supervisory Responsibility

The salary of sanitarian directors varies quite directly with the number of professional persons supervised. This is shown in Chart XX. From Chart XI,

SANITARIANS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XV

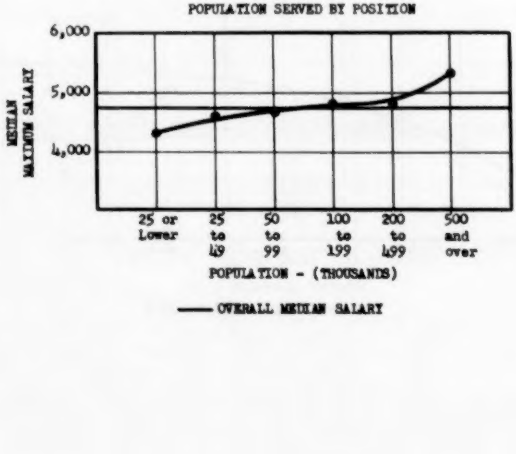
NUMBER OF PROFESSIONAL PERSONNEL SUPERVISED



SANITARIANS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XVI

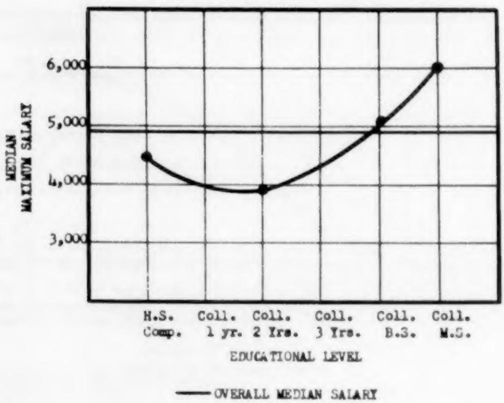
POPULATION SERVED BY POSITION



SANITARIAN DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XVII

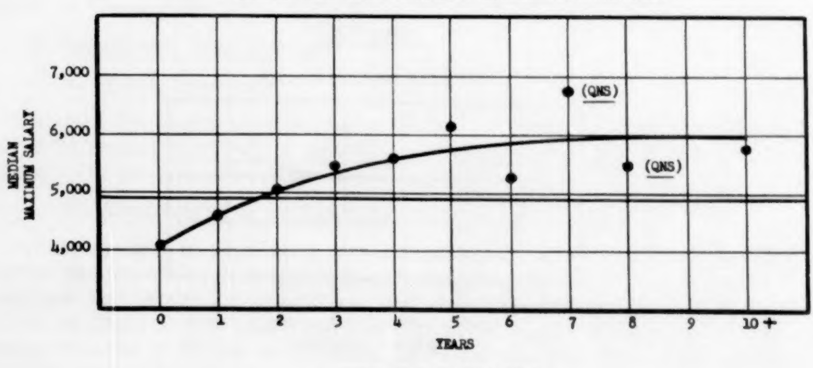
EDUCATIONAL LEVEL REQUIRED FOR APPOINTMENT



SANITARIAN DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XVIII

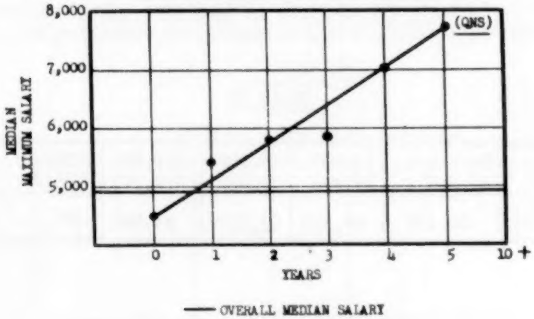
MINIMUM TOTAL YEARS EXPERIENCE REQUIRED FOR APPOINTMENT



SANITARIAN DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XIX

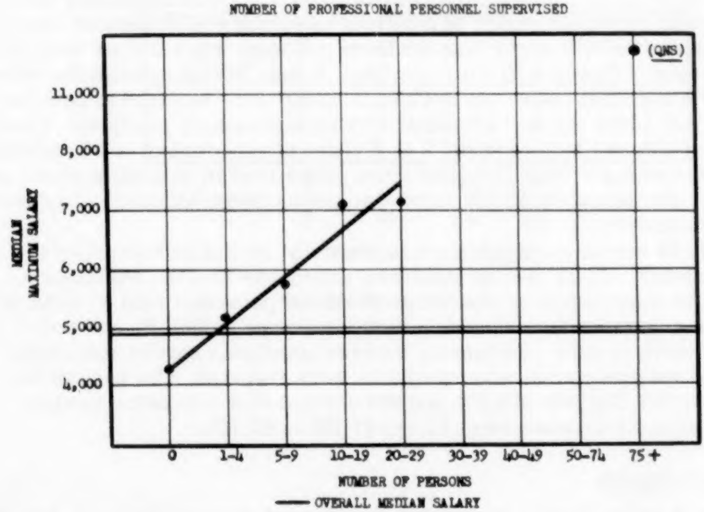
MINIMUM YEARS ADMINISTRATIVE OR SUPERVISORY EXPERIENCE REQUIRED FOR APPOINTMENT



SANITARIAN DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XX

NUMBER OF PROFESSIONAL PERSONNEL SUPERVISED



which provides similar data for engineer directors, it is evident that the salary of engineer directors is not nearly so well related to the number of professional persons supervised.

4. Population Served

Like engineer directors, sanitarian directors are paid in relation to the population served by the department. This is reflected in Chart XXI.

Median maximum salary levels of sanitarian directors were as follows:

TABLE 14

Population Served	Less than 25,000	25,000 49,000	50,000 99,000	100,000 199,000	200,000 499,000	500,000 and over
Med. Mas. Sa.	$4,105	$4,520	$5,310	$5,860	$7,125	$10,000 (Ave.)

SECTION IV - Other Sanitation Personnel

A. Veterinarians

There were 99 veterinarian positions in local environmental sanitation divisions reported upon in this study. They all were required to have obtained the D.V.M. degree. Salaries reflected total experience requirements in a general way. There was considerably more variation here than among engineers and sanitarians.

One-third of all the positions required registration or eligibility therefor. The median maximum salary of positions requiring registration or eligibility therefor was $900.00 lower than for those positions which did not have this requirement. This is difficult to explain. It may be that more of the veterinarians doing meat inspection work exclusively were required to have registration than those who had administrative or supervisory positions. There were 8 positions which required 2 or 3 years of supervisory or administrative experience for eligibility, and these paid a median maximum salary of $8,250 - approximately $2,000 more than the median maximum salary for all veterinarians.

Of the 99 veterinarian positions in the study, 55 had no supervisory responsibility. Their median maximum salary was $6,015. Positions requiring the supervision of 10 to 19 professional personnel paid up to $9,250. Except for veterinarians serving population groups of 500,000 and over ($7,095) there is little relationship between population served and salary.

Three veterinarian director positions were reported. One each in the 50,000-99,000, 200,000-499,000 and 500,000 and over population groups. Their maximum salaries ranged from $7,750 to $9,750.

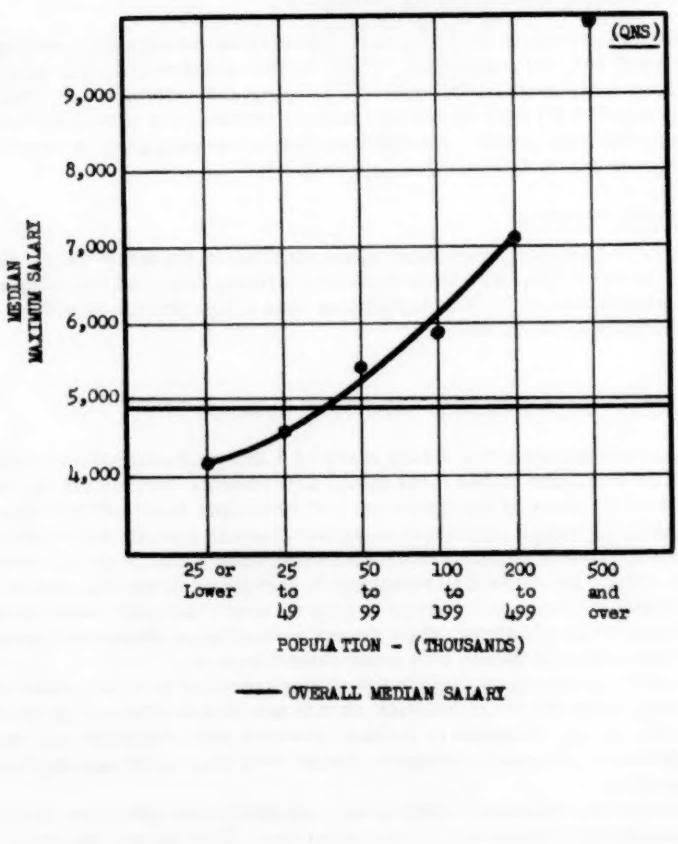
B. Entomologists

Of the 5 entomologists reported in the study, 4 were employed in departments serving populations of over 500,000. One was reported in a department in the 50,000-99,000 population range. Four entomologist positions required a bachelor's degree and 1 required a master's degree. The maximum salaries

SANITARIAN DIRECTORS IN LOCAL HEALTH DEPARTMENTS - 1956

CHART XXI

POPULATION SERVED BY POSITION



for the 4 positions requiring a bachelor ranged from $5,750 to $7,750. The position which required a master's had a maximum salary of $6,750.

C. Lay Health Officers

Eighteen lay health officer positions were reported in the study. Of these, 12 were in departments serving populations of less than 25,000, 4 and 2 served departments in the 25,000-49,000 and 50,000-99,000 population range respectively. Salaries increased somewhat with the larger population groups. Ten lay health officers had no supervisory responsibilities. There appeared to be some relationship between number of persons supervised and salary. The median maximum salary for all was $4,750.

D. Vector Control Foremen

Eleven vector control foremen were reported by departments serving populations of 100,000 and higher. Their median maximum salary was $5,200. Two of these were required to have had a college education, 1 was required to have completed 2 years of college, and the remaining 8 were required to have completed high school. Educational requirements appear to bear a minor relationship to the salaries reported.

E. Plumbing Inspectors

Thirty-four plumbing inspectors were reported in the study with a median maximum salary of $5,520. Most of these positions required the completion of a high school education, 3 accepted less than a high school education, and 2 required a college education.

SUMMARY: SECTIONS II, III and IV

This is a continuation of a salary study of 3,251 professional environmental sanitation positions in 371 local health departments carried out by the Committee on Salaries of the Conference of Municipal Public Health Engineers.

Sections II, III and IV represent an effort to relate educational requirements and experience, population served, supervisory and program responsibility with salary levels paid to positions in various professional groups.

Educational requirements appear to play an important part in the salary level of engineering positions, while among sanitarian positions educational requirements relate to salary to a much lesser degree.

While salaries among engineering positions increase generally with experience requirements of the position, this is not true for sanitarian positions.

There is a direct relationship between years of administrative and supervisory experience required and salary among both sanitarian and engineer director positions.

The salaries of engineers, sanitarians, and sanitarian directors increase with the number of persons which they supervise. This is true only to a much lesser extent among engineer directors.

The popularion served by the position appears to be a factor closely related to the salary of engineer, engineer director and sanitarian director positions but not among sanitarian positions.

The report also provides data on types of degree required, registration, and program specialization. The report provides some information concerning

the salaries of veterinarians, entomologists and other personnel employed in environmental sanitation divisions in local health departments.

The subsequent and final Section of the Report will present the conclusions and recommendations of the Committee.

Note: For those who wish to use this Report for comparative purposes or for the setting of salary levels, the Committee wishes to point out that these data were collected during June 1956 and that salary levels for the groups studied have been rising at rates varying between 5% and 13% each year.

COMMITTEE ON SALARIES, Conference of Municipal Health Engineers

Walter A. Lyon Chairman
William T. Ballard
Herbert J. Dunsmore
Reinhart W. Koch
John W. Lemon
Eric W. Mood
Louis W. Pickles
Jack C. Rogers
Lester A. Sanger

Paper 1686 June, 1958 SA 3

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

A STUDY OF SEWAGE COLLECTION AND DISPOSAL IN FRINGE AREAS
Second Progress Report of the Committee on Public Health Activities of the Sanitary Engineering Division
(Proc. Paper 1686)

INTRODUCTION

In this Second Progress Report, the Committee presents Appendices B, C, D and E to supplement the First Progress Report published as Proceedings Paper No. 1613 in the April 1958 Journal of the Sanitary Engineering Division.

A discussion of experiences in Los Angeles, California, appeared as Appendix A in the First Progress Report. Herein are presented, the experiences at Seattle, Wash., Houston, Tex., Kansas City, Kan., and Kansas City, Mo. Appendix E contains some suggestions for implementing the master plan and for making greater use of the interim facilities to permit the master plan to be completed in a reasonable time.

Appendix B.—Experience in Seattle, Wash.

Seattle is located in King County, Washington and is the largest city in the state. First settled in 1851, it was incorporated in 1865. In 1910, 837 of the population of King County resided in Seattle. Prior to 1940 very little development occurred outside the city and only two annexations were made to the city between 1910 and 1940. After 1940 the growth pattern changed rapidly. Seattle was the 29th fastest growing standard metropolitan area (SMA) in the United States during the 1940-50 period. The growth was largely outside the central city and by 1950 the percentage of King County population living in Seattle had dropped to 64 per cent as compared to 85 per cent in 1910.

Table I shows the estimates of the King County and Seattle Planning Commissions with regard to anticipated growth of various areas.

The following excerpt from Research Bulletin #14, Seattle City Planning Commission, dated April 1, 1954 points to one of the basic reasons for fringe area growth that, "Careful inventory of the buildable residential lots in Seattle

Note: Discussion open until November 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1686 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 3, June, 1958.

Table I

Area	Population 1955	Population 1970
Seattle City	555,000	580,000
Greater Seattle (City & Suburbs)	716,000	877,000
Remainder King County	79,000	141,000
Total King County	795,000	1,018,000

indicated in 1950 that only 8 or 9 out of 100 home sites remain vacant within the city limits (as of 1950). A recheck in 1953 showed that in three years 1 out of 5 of the 1950 vacant lots had been occupied by homes. The areas nearest to the city are, in a like manner, approaching full development. This explains the marked slowing of growth in the city itself, (1950 limits) the relative slowing in the areas closest to the city, and the great increase in population projected for the residential areas which lie further from the city center".

Several areas outside the city proper show an increase in population from 1940 to 1950 of more than 25%.

Table II shows the number of building permits issued in King County from 1950 through 1955. These figures are taken from records of the King County Building Department and do not include any permits issued within the Seattle city limits. Thus, these figures are for the Seattle SMA, exclusive of the central city.

Table II

King County Building Permits

Year	A Total Permits	B Single Family Residence	C Other Buildings with Plumbing	D Buildings without Plumbing
1952	7138	4746	270	2124
1953	7009	4397	414	2198
1954	7848	5612	332	1904
1955	8073	5660	350	2063

Columns A and B are self-explanatory. Column C includes buildings with plumbing which would contribute to the total sewage load. These include commercial buildings, schools, duplexes, apartments, auto courts, churches, fire stations, service stations, and similar structures. Column D includes buildings which do not have plumbing such as additions to present structures, garages, barns, sheds, and advertising signs.

It will be noted that the number of new buildings constructed outside the new city limits continued to increase despite the annexation in 1954 of several large areas (Fig. 1), including the rapidly growing area to the immediate north of the 1950 city limits. This is indicative of the growth of the Seattle area.

The Seattle SMA is an area with higher than average percentage of owner-occupied dwellings. Even the central city shows more than 56% of the dwellings to be owner-occupied. Table III is taken from the 1950 U. S. Census of

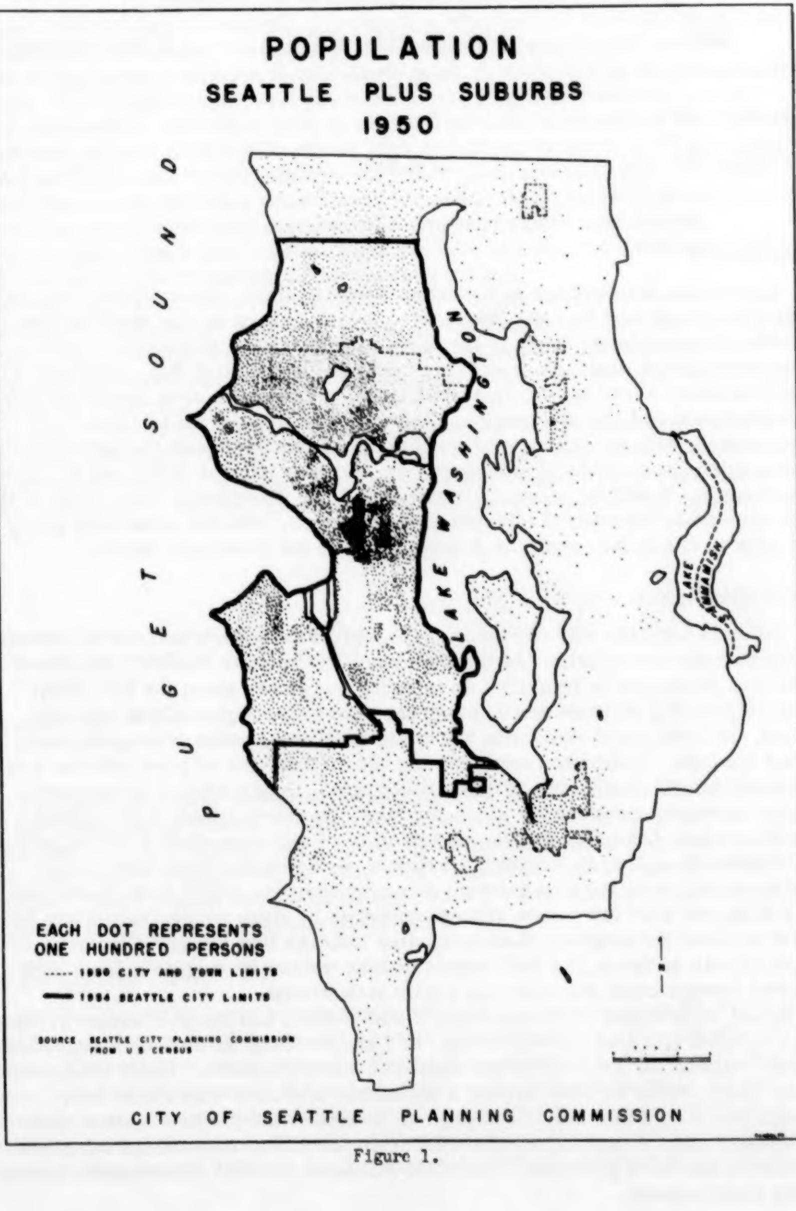

POPULATION
SEATTLE PLUS SUBURBS
1950
PUGET SOUND
LAKE WASHINGTON
LAKE SAMMAMISH
EACH DOT REPRESENTS
ONE HUNDRED PERSONS
1950 CITY AND TOWN LIMITS
1954 SEATTLE CITY LIMITS
SOURCE SEATTLE CITY PLANNING COMMISSION
FROM U S CENSUS
CITY OF SEATTLE PLANNING COMMISSION

Figure 1.

Housing and shows the number of dwelling units and the percentage owner-occupied in the area.

Table III

The 1950 U. S. Census of Housing reports the following:

Area	Occupied Dwelling Units	% Owner-occupied
Seattle City	154,582	56.5%
Seattle Urban Area	201,506	61.0%
Seattle SMA	236,258	63.2%

Topography

Generally, the drainage in the urban area is either west to Elliott Bay and Puget Sound or east to Lake Washington, which in turn drains west through the Canal and government locks to the Sound. To the south of Seattle lies the Duwamish River drainage area, which empties into Elliott Bay. Just east of the Duwamish River basin lies the Cedar River drainage area emptying into the south end of Lake Washington. East of Lake Washington is Lake Sammanish, with its drainage area. Lake Sammanish drains northward to the Sammanish River (slough), which in turn, empties into the north end of Lake Washington. It will be noted therefore that Lake Washington takes most of the drainage from the central city and the urban area, with the remainder going directly to the Sound, or to the Sound by way of the Duwamish River.

Lake Washington

Prior to 1922 the city of Seattle discharged raw sewage to Lake Washington from 33 separate outfalls. At the insistence of the State Health Department a plan was developed to construct an intercepting sewer along the lake front with 12 pumping stations and to pump the sewage through outfalls into the Sound. In 1939, rigid standards were laid down for sewage treatment plants along the lake. Complete treatment and the disinfection of plant effluent was required for all discharges to Lake Washington. Since 1940, 11 treatment plants have been constructed along the lake. However, many storm drains discharge into Lake Washington, and in spite of vigorous efforts on the part of the Seattle-King County Health Department, many hundreds of homes contribute pollution to the lake through direct or indirect septic tank discharges.

During the past few years, a large increase in algae concentration has been noted in Lake Washington. Recent studies indicate that the great increase in algae growth is due to the enrichment of lake waters by nutrients from both treated sewage plant effluents and septic tank drains.

Health authorities, pollution control authorities, and many citizens' groups have become alarmed at the prospect of Lake Washington becoming degraded to uselessness for recreation and domestic water supplies. The State Health Department, early in 1956, issued a Statement of Policy with respect to sewage discharge into Lake Washington. In effect this policy requires that all sewage must eventually be diverted from the Lake, even though complete treatment has been provided. The State Pollution Control Commission issued a like requirement.

These policies caused a new look to be taken at the handling of domestic sewage in the entire urban area, since most of the area drains to Lake Washington.

Sewage Disposal in Urban Areas Around Central City

As areas around the central city build up, the practice has been to utilize septic tank systems for the disposal of sewage from residential and commercial buildings. All such installations must be approved by the Seattle-King County Health Department, and all construction requires a building permit from the County. Several extensive areas around the central city have ground conditions of such a nature (hardpan, high ground water table, etc.) that septic tank systems do not function unless very large lots are available. In these areas, the County Building Department refuses to issue building permits until the health department first approves the proposed septic tank system.

Generally, when septic tank systems produce trouble in a developed area, a sewer district is organized to handle the problem.

Practically all of the sewer districts in the urban area were formed to correct serious sewage disposal problems resulting from such improperly functioning private systems. It has been almost impossible to promote the formation of sewer districts before the problem becomes so acute that it forces the residents of the area to petition for the district.

Table IV shows the number of building permits issued outside the central city from 1952 through 1955 and the number of septic tank permits issued during the same period. Although some of the septic tank permits were issued for corrections to existing faulty system, the majority serve new construction.

Table IV

	1952	1953	1954	1955
Permits issued for buildings contributing sewage in areas outside central city.	5016	4811	5944	6010
Permits issued for septic tank installations outside central city.	4134	4137	5179	6171

The problem of septic tank systems requires considerable time on the part of the Seattle-King County Health Department. In 1952, 19,098 separate inspections of individual sewage disposal systems were made by personnel of the local health department; in 1953, 22,225; in 1954, 23,824; and in 1955, 25,832. These inspections included approvals for new installations and investigation of complaints on existing installations.

Sewer District Laws

Washington State law permits the construction of sewer systems under the Dyking, Drainage and Sewage Improvement District Act of 1913, herein the County Commissioners become the Sewer Commissioners; the County Engineer becomes the engineer for the District; and provision is made for the system to be constructed and maintained by the County. Several features of this law have proved undesirable. Although the taxpayers paid the costs of the system, they had no voice in the development of plans, method of financing, or

management. No consideration was given to the future growth of the area.

With the increase of population in suburban areas and the demand for better sanitation, new laws were necessary to enable these areas to construct and maintain their own sewer systems. At the present time, a sewer district is organized by the voters living in the area who elect their own commissioners to conduct its affairs. It is based upon the wishes of the majority of the people. A comprehensive plan for present and future needs is required, and before any construction can begin, these plans must meet the approval of the state sanitary officials.

In general, a sewer district is formed by first obtaining a petition signed by at least 25% of the qualified voters in the proposed district. The petition is then filed with the County Auditor and certified to the Board of County Commissioners, who set a hearing date to define proposed boundaries. After the hearing, an election date is set. In order for the district to be formed (1) it must receive at least 60% of the votes cast, and (2) this number must constitute at least 40% of the total votes cast at the last general state election. Three sewer commissioners are elected at the same time, and a one-year levy, not to exceed five mills of assessed evaluation, is established for general preliminary expenses. The sewer district commissioners are elected for a six-year term. They employ an engineer to prepare comprehensive sewerage plans for the district.

In order to construct the system, bonds may be issued. These may be general obligation bonds, revenue bonds, or a combination of these. Local improvement utility districts are formed in order that funds can be raised by assessment.

General obligation bonds are limited to 5% of the assessed valuation, they are generally insufficient to meet the entire cost of construction, and such money must be used for the general benefit of the entire district. Straight revenue bonds can be issued which provide for no assessments. The entire cost of the bonds is paid by income derived from service charges.

The combination of bond issues is the most generally adopted form of financing. This may include general or revenue bonds, or a combination of general and revenue bonds supported by a combination of local improvement utility district assessments and service charges.

There are at present in King County 11 active sewer districts serving populations ranging from 400 to 30,000; 9 inactive districts, 12 miscellaneous systems (serving shopping centers, industrial plants, etc.), and 7 municipal systems. Most of these districts and systems fall within the urban area of Seattle. The operative sewer districts are shown in Table V.

Planning Efforts

A report dated August 28, 1956, prepared by the Sanitation and Pollution Committee of the King County Planning Commission, presents a proposed plan to provide a sewerage system for what is known as the Highline District of the county. This embraces an area from the south Seattle city limits, halfway to the south King County limits; and from Puget Sound east to the Duwamish River basin. This area includes some seven existing sewer districts, and a large area not included in any district.

While the report is confined to only a part of the SMA, it does indicate a trend in thinking toward some overall approach to the sewage problem, and as do most of the other recent reports of this type, it points out the inadequacy of trying to solve sewage problems by the present methods.

Table V. SEWER DISTRICTS IN KING COUNTY (Operative)

	Formation Date	Area (sq.mi.)	% of potential sewer area	Treatment Design Population	Pop. Served 1955	Age of Plant	% of capacity	Type of Treatment
SEWER DISTRICTS								
Lake City	1946	12.20	95%	25,000	25,000	6.0	125%	Activated sludge, diffused aeration, chlorination.
Greenwood Ave.	1946	2.70		25-30,000	25,000	6.0	75%	Fine grinding, disintegration of solids, removal of floatable material and disinfection.
Lake Hills	1955	1.88	2%	400		1.0		Imhoff tanks, primary settling, chlorination.
Bellevue	1948	6.10	25%	8,500	5,000	2.0	65%	Activated sludge.
Mercer Island	1953	.71						Pump to Bellevue Trunk
East Mercer	1953	.28	100%	1,000	320	1.0	30%	Intermediate: activated sludge, mechanical aeration.
Bryn-Mawr - Lake Ridge	1950	.75	75%	4,000	4,200	3.0	100%	Primary settling, activated sludge, separation sludge, digestion and chlorination.
Roxbury Heights	1944	.75				10.0	125%	Imhoff tanks.
Southwest Suburban	1945	7.10						Primary
Val Vue	1945	.20	5%	2,500	1,400	0.5	40%	Coarse screening, grit removal, primary sedimentation, sludge digestion and chlorination.
Vashon	1947	.38	75%	500	300	1.0	60%	Imhoff, filtering, settling and chlorination.
SEWERAGE & DRAINAGE DISTS.								
Sewerage & Drainage Improvement District #3		.19	100%	1,000	800	13.0	100%	Underground cominutor wrecked by sand and gravel; chlorination abandoned.
Sewerage & Drainage Dist.#4		.35	100%	2,400	2,500	13.0	100%	Primary; Imhoff tank, chlorination.

In the past few years, the rapid growth of the urban area, and the attending sewage disposal problems, has resulted in many groups considering means of correcting the situation. Such groups as the King County Planning Commission; the State Health Department; the State Pollution Control Commission; the Bureau of Governmental Research; and many other official and non-official agencies, as well as interested lay groups, community clubs, etc., have talked about a metropolitan sewer district. The State Health Department and the State Pollution Control Commission have studied the problem to some extent, and have presented reports, with general recommendations toward a metropolitan district. These reports and many others have been only general, however, and have not been sufficiently detailed so as to provide a concrete base upon which to build such a district. State enabling legislation is needed to legally set up any kind of a metropolitan sewer district which would be of any help in solving the problem. So far, efforts to secure such enabling legislation have failed. The difficulty stems from a reluctance on the part of any of the existing political subdivisions to relinquish any of their rights and prerogatives to a metropolitan board.

In September 1952, the State Pollution Control Commission issued "Technical Bulletin No. 13", titled "The Sewage Disposal Problem in the Seattle Metropolitan Area". This report divided the suburban area into 14 proposed sewer districts, based upon natural drainage boundaries, existing sewer districts, treatment plant sites, and predicted population concentrations. Recommendations were made for a comprehensive sewerage plan, an overall coordinating agency and improvements in existing sewer district legislation.

Many general plans and ideas have been advanced by many groups and agencies, but there has been no coordination of these ideas and plans into a real workable solution.

A recent formation of Lake Hills Sewer District has brought into sharp focus the real problem of sewage disposal, and the need for a much broader approach to solving the problem. A brief report on this district with its problems will serve as an illustration of what appears to be the beginning of a change in public attitude toward the problem.

Early in 1955 R. H. Connor and Associates, who had recently developed a subdivision known as Eastgate, obtained control of an area (Lake Hills) suitable for an ultimate 4,000 lots and proposed to develop the property. After consultation with the Seattle-King County Health Department and their own consulting engineers, it was decided that much of the area was not satisfactory for septic tank disposal systems, unless very large lots were provided. In order to utilize the land more economically, it was decided that a central sewer system should be planned. Accordingly, the Lake Hills Sewer District was formed in August 1955. Because of existing state statutes it was necessary to secure a petition and hold an election. At the time of this action there were six voters in the area; actually these were employees of the developer. Following the formation of the district, a consulting engineer was employed to prepare a comprehensive plan for the district in September 1955. The comprehensive plan for the sewerage system was drawn up and received the necessary approval of both the State Department of Health and the State Pollution Control Commission in October 1955.

This comprehensive plan called for a temporary sewage disposal system consisting of an Imhoff tank and irrigation with the effluent. The plan also called for a permanent disposal system consisting of secondary treatment and final discharge to the Sammanish River (or slough) just north of Lake

Sammanish. The permanent treatment plant site was approved by the state agencies and was secured by the district.

Some estimates of costs are of interest. In the Eastgate Subdivision (1500 lots) where septic tanks have been provided, the average cost per house was approximately $250 to $300. The estimated average cost per lot for the sewage system for the Lake Hills development is approximately $325. This includes the present temporary treatment plant. The cost of the permanent plant will be financed by the $3.00 per month service charge. The Eastgate Subdivision will eventually be served by a central sewer system, and the monthly service charge will probably be the same.

It has been said by the developer that the slight additional cost for the central sewer system is more than offset by the advertising value of "no septic tanks" and "no future assessments".

In January 1956, the State Pollution Control Commission advised the Lake Hills Sewer District that it was planning to reconsider its previous approval as to the final sewage disposal plans. This action on the part of the State Pollution Control Commission was the result of several factors, the most important being (1) increased public opposition to pollution of Lake Washington; (2) a proposal on the part of the Bellevue Sewer District to remove its effluent from Lake Washington and pipe it around the south end of the Lake to the Duwamish River; and (3) in order for the Bellevue Sewer District proposal to be economically feasible, it would be necessary to include Lake Hills and other districts.

A series of meetings were held and it was apparent that the success of the Bellevue Plan rested upon Lake Hills being willing to join the system. However, the cost of the Bellevue Plan to the Lake Hills residents would greatly exceed the stated $3.00 per month service charge already agreed upon by the Lake Hills Sewer District Commission. From February to June 1956, meetings were held but no agreements could be reached. To further complicate the problem, the F.H.A., which had been brought in at the beginning of the Lake Hills Development and had endorsed the sewer plans, found that unless Pollution Commission approval could be restored, they could no longer make loan commitments.

The Lake Hills Sewer District petitioned the State Pollution Control Commission to reinstate their approval of their comprehensive plan. At the hearing on this petition, the consulting engineer for the Lake Hills Sewer District presented an alternate plan for handling the sewage without increasing the cost of the Lake Hills residents. It is another comprehensive area approach to the whole urban area, and appeared to present some more favorable factors than did the original Bellevue overall plan.

As a result the Commission went along with the revised Lake Hills plan.

This controversy pointed up the real need for handling sewage problems on the metropolitan basis. It pointed out also that although the State Pollution Control Commission had legal authority to approve or disapprove comprehensive plans, it is not equipped to provide the overall studies needed, or the authority to set up a plan and require future developments to fit into it.

Two consulting engineering firms presented plans for handling sewage on a metropolitan basis. Because no official agency had the legal responsibility to set up a master plan, the issue quickly became a difference of opinion between two highly regarded professional engineering concerns.

Because the Lake Hills proposal seemed to be better substantiated by cost figures more favorable, and because it proposes ultimate discharge to deep

tidal waters of Puget Sound rather than the Duwamish River, it was considered the best of the two proposals.

At any time, a third or fourth consulting engineering firm may come forth with proposals that could be considered even more practical, and the controversy would again develop.

The need for an authoritative overall planning agency to prepare a master plan for sewage disposal in the urban area, and including the central city, becomes more and more apparent.

Governor's Committee on Metropolitan Problems

Early in June 1956, the Governor of the state of Washington, recognizing that the continued growth of areas adjacent to central cities presents a number of problems seriously affecting orderly development, set up a committee of approximately a dozen people representing city, county and state government, the state legislature, and several municipal associations and organizations. The objectives of the committee were to serve as a coordinating group, to study the various problems as they were presented, and recommend practical solutions, particularly in the area of needed state legislation.

In setting down a list of the problems confronting orderly development of metropolitan areas, the problem of sewage was considered top priority. In the several meetings already held this year, the Governor's Committee has continued to discuss the problem of sewage disposal, particularly in the Seattle Metropolitan Area. Although no official action has yet been taken by the Committee, preliminary reports of the meetings indicate a feeling that a complete engineering study is needed for the Seattle area, and discussions have centered on means of financing such a study, with the suggestions that the various governmental agencies concerned, including state, county and municipal, be asked to contribute toward the anticipated cost. At one of the recent meetings, a number of local consulting engineering firms, as well as several municipalities, offered to provide information and reports already in existence, to any person selected by the Committee to undertake the overall study.

Present Action

The city of Seattle recently employed an outside sanitary engineer as an expert to study the sewage problem within the city. The people of the city recently noted a sewer assessment charge to provide treatment for the city's sewage. This assessment was favorably voted by more than a two to one majority, and indicates the interest of the people in removing pollution from Lake Washington and the nearby Sound beaches. The county followed the city's example and contracted with the same engineer to broaden his study to the surrounding urban area.

This action of the city and county, while not a direct result of the preliminary reports of the Governor's Committee, is expected to work into that committee's recommendations.

The Municipal League of Seattle recently published a report recommending a metropolitan approach to many of the city and county functions now carried on separately.

A much more elaborate report is now being prepared by the Bureau of Governmental Research at the University of Washington along the same lines.

The problem of sewage disposal is considered in both these reports. However, both groups feel that a number of other functions, such as library facilities, fire protection, and the like, need to be considered on a metropolitan

basis as well as sewage disposal. The Bureau of Governmental Research report will indicate that one metropolitan board could administer the several governmental functions now handled by the several cities, towns, and the county.

At this time it would seem less likely that such a proposal will receive widespread approval than would a metropolitan district set up solely for sewage collection and disposal. Undoubtedly, as time goes on the more comprehensive plan will be more acceptable than it is anticipated it will be at the present time.

Appendix C.—Experience in Houston, Tex.

The Houston Standard Metropolitan Area, as defined by the Bureau of Census, includes all of Harris County, Texas. Harris County leads all others in population, manufacturing, industry, and rail and water shipping. It is located on the coastal plains of southeast Texas facing Galveston Bay. Houston is the county seat of Harris County and is Texas' most populous city. In addition to Houston, there are 15 other incorporated cities in the county. These incorporated cities range in size from Shoreacres with only a few hundred people to Pasadena with an estimated 1955 population of 39,000.

Harris County is rather old, and it has experienced a steady growth for 100 years:

Year	Population
1850	4,668
1860	9,070
1870	17,375
1880	27,985
1890	37,249
1900	63,786
1910	115,693
1920	186,667
1930	359,328
1940	528,961
1950	806,701
1956, Jan. 1 Estimate	1,075,000

The population density in 1950 was 466.3. On the basis of the January 1, 1956 estimate of 1,075,000 population, the population density is approximately 620 persons per square mile.

The trend in subdivision development in Harris County has been away from the central city. In spite of the annexation of new territory, most of the new development is still outside the corporate limits and as shown in the following table reporting 10 years of development.

In 1955, 76 per cent of the new building sites were outside the corporate limits of Houston.

Methods for Providing Sewerage Systems

The housing developer or public official faces choices for providing sewerage services to the subdivisions and developments. Individual sewage disposal

Year	Approximate Number of New Lots	Within City Limits		Outside City Limits	
		Number	Per Cent	Number	Per Cent
1945	3,900	750	19.3	3,150	80.7
1946	9,500	2,200	23.2	7,300	76.8
1947	8,700	1,600	18.4	7,100	81.6
1948	8,600	800	9.3	7,800	90.7
1949	13,500	1,000	7.4	12,500	92.6
1950	19,000	10,000	52.5	9,000	47.5
1951	8,000	3,500	43.7	4,500	56.3
1952	14,000	6,500	46.5	7,500	53.5
1953	15,000	3,750	25.0	11,250	75.0
1954	20,000	5,500	27.5	14,500	72.5

systems in the form of septic tanks and absorption fields are often considered first. However, such systems have not been satisfactory and methods are sought whereby community sewer systems and treatment works can be provided. In Texas, three methods are available for providing such services. They are (1) incorporation, (2) water control and improvement districts, and (3) private stock companies. These three methods will be discussed in the following pages. There is one exception that applies only to Harris County. The legislature has authorized Fresh Water Supply Districts in Harris County to issue bonds for the construction of sewerage facilities. This applies to no other county in Texas.

Incorporation

The incorporation of a community in Texas permits it to undertake certain local activities including the construction and operation of a sewerage system. State Law authorizes cities and towns to finance the construction of a sewerage system by the issuance of bonds supported by a tax on the property, bonds supported by a pledge of the revenue derived from service charges, or by a combination of tax supported and revenue bonds.

The attitude of the bond dealers and buyers, in general, has been such that revenue bonds alone do not constitute a feasible method of financing a sewerage system. However, if the water and sewerage facilities are constructed as a combined project, revenue bonds based upon both water and sewerage service charges are marketable. Proposed revenue bond issues must be submitted to the qualified voters at an election, and the bonds approved by the State Attorney General.

Tax bonds are a common method of financing public improvements. Tax bonds may be issued only after the proposal has been approved by the qualified voters who are property taxpayers of the city or town. The bonds must be approved by the State Attorney General before they are offered for sale.

Water Control and Improvement Districts

Communities may develop sewerage systems through the formation of Water Control and Improvement Districts. The district may be organized under the Commissioner's Court of a County. If the district is included in two or more counties, it is organized under the State Board of Water Engineers.

The procedure for organizing a Water Control and Improvement District is as follows:

(1) The petition shall include the following:

(a) Set the boundaries of the district.
(b) Give the general nature of the work proposed to be done by the district.
(c) State the necessity of the district.
(d) Declare the feasibility of the district.
(e) Designate a name for the district.
(f) State the provision of the constitution under which the district is to be organized.
(g) State the estimated cost of the project as estimated by those filing the petition.

(2) The petition shall be signed by a majority of the landowners of the district and the owners of the majority in value of the land in the districts. If the number of landowners in the district is more than fifty, the petition must be signed by fifty landowners.
(3) The petition shall be filed in the office of the County Clerk of the county in which the district is situated. The County Judge makes an order setting a hearing on the petition by the County Commissioner's Court. The County Clerk issues notice of the hearing; delivers a copy of the notice of the hearing to the Sheriff, who posts it at the courthouse door fifteen days prior to the date of the hearing; and publishes one copy in a newspaper of general circulation in the county.
(4) If the petition is granted by the Commissioner's Court, the court shall appoint five directors who shall qualify and serve until their successors are elected or appointed.
(5) After the directors have qualified as stated above, the directors shall meet and elect officers and enter upon the discharge of their duties as follows:

(a) Within thirty days after the date of the first meeting of the directors, the directors shall make and publish an order calling an election within and for the district.
(b) The notice of the election shall state the purpose of the election, the propositions and officers to be voted upon and shall be published as provided by law.
(c) The qualified resident property taxpaying voters vote for the confirmation of the district or against the district, vote upon the directors, and vote for or against the preliminary bonds.
(d) If a majority of those voting at such election vote in favor of the confirmation of the district, the organization of the district is confirmed and ratified.

(6) The directors elected and qualified, as stated above, enter upon the discharge of their duties, which include the employment of an engineer to prepare plans, specifications, and cost estimates.

Financing by Water Control and Improvement Districts is much the same as financing methods employed by incorporated towns and cities. The district may issue tax bonds, revenue bonds, or a combination of both. Both types of bonds must be approved by the Attorney General before they are offered

for sale. The bond issues are submitted to the qualified voters of the district and if favorable consideration is given, the Board of Directors of the district may issue the bonds.

Private Stock Companies

State laws make provisions for the creation of private corporations for the purpose of constructing and maintaining sewers. These corporations may be created by the voluntary association of three or more persons for the purpose of constructing and maintaining sewers. These corporations are usually referred to as public utility companies and operate much the same as the gas, electric, or other public utilities.

Sewerage System Development in Harris County

The initial trend in Harris County was toward individual sewage disposal systems employing septic tanks and absorption fields. Such installations, however, do not perform satisfactorily because of heavy rainfall, high ground water table, poor drainage, impervious soil conditions, and low evaporation rates in the winter months. The use of individual sewage disposal systems has been discouraged, and the trend is toward community sewerage systems financed by districts, incorporated towns, or private utility companies.

According to the Harris County Health Department, there are 91 sewage treatment plants in Harris County, outside the city of Houston, serving a total estimated population of 256,000. Twenty of these sewage treatment plants are owned and operated by municipalities and serve an estimated population of 167,000. Fifty-three plants are owned by private individuals or utility companies and serve approximately 52,000 persons; and 18 plants are owned and operated by districts and serve approximately 37,000 persons. These figures were altered somewhat on January 1, 1957, when the city of Houston annexed a large territory served by some of these systems.

The sewage treatment plants of the municipalities are generally larger than those owned by districts or private utilities and vary in size from a design capacity of 1,500 to a design capacity of 40,000 persons. The private plants range in size generally from a design capacity of 100 persons to 3,500 persons. These are, however, some larger plants. The sewage plants serving the districts will serve from 1,000 to 10,000 persons. The smallest plant in Harris County will serve 90 persons.

Three basic types of sewage treatment plants are widely used in Harris County: Contact aeration, activated sludge with either mechanical aeration or diffused air, and the trickling filter. Fifty-one of the plants are trickling filters, 18 are contact aerators, 18 are activated sludge. The remaining 4 provide only primary treatment or use the Dunbar filter as a secondary treatment device.

Approximately 30 of the trickling filter plants are constructed of steel. One manufacturer's package activated sludge plant is used rather extensively.

Harris County has no regulations covering the location of sewage plants or the minimum distances from residences. One contact aeration plant is within about 60 feet of six residences. A number of plants are located within 200 feet of residences.

Sixty-five sewer districts have been organized in Harris County but in 1956 only 37 were active. Some have been taken into the corporate limits of cities and others did not proceed beyond the initial formation.

The districts, both the fresh water supply districts and the water control and improvement districts are organized under state law as previously outlined. They operate much the same as incorporated municipalities.

Privately owned systems appear to be gaining favor in the Houston area. Formerly the Federal Housing Administration and the Veterans Administration ruled that when a privately owned sewerage system was sold, the money derived from the sale of the system had to be returned to the property owners in the area served by the utility. This ruling is no longer in effect.

In financing a private system, the lines are laid in the subdivision, the treatment plant is constructed, and the total cost is prorated among the lots in the subdivision. As the lots are developed, a connection fee is collected, and the utility company collects a sewerage service charge.

In addition to the provision of sewerage services, as described, subdivisions may contract with an adjoining private system or municipality to take the sewage from the area. In this case, the developer installs the collection system and deeds it to the private utility or municipality.

Interim Treatment

The only treatment plants that might be considered to be of interim type are the steel trickling filter plants. Such plants include a circular Imhoff tank, high rate trickling filter, final settling tank, and a chlorine contact tank. With the rising costs of steel, it is doubtful that the use of the steel plant can be economically justified; however, it has the advantage that it can be moved or salvaged when the plant is no longer needed.

There has been some discussion of waste stabilization ponds in Harris County but the land requirements and high land cost have discouraged their consideration for interim treatment.

Operation of Sewerage Systems

The fresh water supply districts and the water control and improvement districts operate their own plants. Some of the private systems are operated on contract. The amount paid for the operation of the systems is a set fee per connection.

Attitudes and Activities of Official Agencies

The city of Houston has no authority in the fringe areas except that vested in the City Planning Commission. The city of Houston, in annexing new areas, assumes the obligations of any district taken in and purchases the privately owned systems. No set patterns are used in annexations. Adjoining areas may be annexed without requiring vote of the people living in the area.

The Houston City Planning Commission has authority under State Law to review the plats of all subdivisions within five miles of the corporate limits and no plat may be recorded by the County Clerk until it has been approved by the City Planning Commission. However, this review does not include planning for sewers.

The County Health Department cannot require a community sewerage system for an area. The Harris County Health Department has, however, done an excellent job in educating developers to the needs and advantages of a community sewerage system.

The State Health Department is concerned with the enforcement of the stream pollution statutes and works closely with the Harris County Health Department. Plans for new sewerage systems must be submitted to the State Health Department for approval.

Federal Housing Administration and Veterans Administration

FHA and VA accept the recommendations and inspections of the Harris County Health Department. Household sewage disposal systems in subdivisions are not approved by the Harris County Health Department.

Summary and Conclusions

The Houston Metropolitan Area (SMA), which includes all of Harris County, has experienced a tremendous growth since 1940. The population has increased from 528,961 in 1940 to 1,075,000 on January 1, 1956. A large portion of the new housing development has been outside the corporate limits of the city of Houston.

The provision of sewerage facilities has been a major problem. In the early years of rapid development, septic tank systems were used. Unfortunately, they do not work well so a trend has developed toward community sewerage systems in the suburban areas. The Federal Housing Administration and Veterans Administration will no longer approve individual sewage disposal systems in subdivisions.

Sewerage systems are provided outside the corporate limits by two methods: (1) Water Control and Improvement Districts and Fresh Water Supply Districts, and (2) private utility companies. There are 51 private sewage plants and 29 plants operated by districts in Harris County. These two methods have been quite effective in improving conditions in the fringe areas. However, problems in the development of these systems remain to be solved, particularly plant location and effluent disposal.

Even though the methods employed in Harris County have been rather effective, it is recognized that these are not the final answer. Engineers, planning officials, and others concerned are of the opinion that the final answer will be a Metropolitan Sanitary District. Such a district was created by the State Legislature in 1953, but as yet, the district has not been activated.

Appendix D.—Experience in Kansas City, Kan. and Kansas City, Mo.

The Kansas City metropolitan area which includes four counties, Clay and Jackson in Missouri and Johnson and Wyandotte in Kansas, is the 17th most populous SMA in the United States. The population of the SMA is slightly more than 814,000 of which approximately 456,000 or 56% live in the central city, Kansas City, Missouri.

The variety and complexity of the legal, economic, and cultural problems that confront engineers, developers and planners in this bi-state multi-county area are typical of these in other SMA's in the U. S. A brief description of the practices being used in this area may indicate some possible approaches to the problem in other metropolitan areas.

Demography

The greatest growth in metropolitan areas is occurring in the outer unincorporated areas of the urban fringe of the central cities. This is true in the Kansas City area as indicated in Table I.

Table I

Area	Population		Percentage Increase
Kansas City SMA	686,643	814,357	18.6
Clay County, Missouri	30,417	45,221	48.7
Jackson County, Missouri	477,828	541,035	13.2
Johnson County, Kansas	33,327	62,783	88.4
Wyandotte County, Kansas	145,071	165,318	14.0

Another statistic relative to the population density will be of interest. The average population density in the central city is 5,665 persons per square mile in the urban fringe of the central city 3,534 persons per square mile, in the whole SMA 229 persons per square mile, and in the outer rural fringe area less than 150 persons per square mile. From this data it is clear that the only place residential growth can occur is in the peripheral areas of the urban fringe. Such areas are of course unincorporated and without essential sewerage facilities. While the spilling over into the fringes started in this area shortly after World War I, it really did not reach full swing until after 1935 when certain programs of the Federal Housing Administration made it possible for developers and builders to construct and finance homes on a subdivision basis, rather than on the old custom-built individual home basis.

Topography

The central city lies in three river basins, the Missouri, the Kansas and the Blue, and tributaries of these streams extend far out into the fringe areas. (See Fig. 1). The more important of these tributaries are (1) Turkey Creek, (2) Brush Creek, and (3) Indian Creek. The Blue River flows through the central city and serves as a boundary between Independence, Missouri, the county seat of Jackson County, Missouri, and the central city. The Kaw River separates Kansas City, Kansas, and Kansas City, Missouri, in the industrial area. Rosedale, a section of Kansas City, Kansas, and Kansas City, Missouri, is separated only by the street "State Line". Across the Missouri River to the north in Clay and Platte Counties, large industrial and residential areas have developed. This area lies for the most part in the Missouri River flood plain and a high ground water table exists in most of the area.

Planning Efforts

Economic, cultural, and political rivalry has split this metropolitan area into many competing sectional groupings. This inter-community rivalry is reflected in planning and zoning efforts. There are at present well-staffed

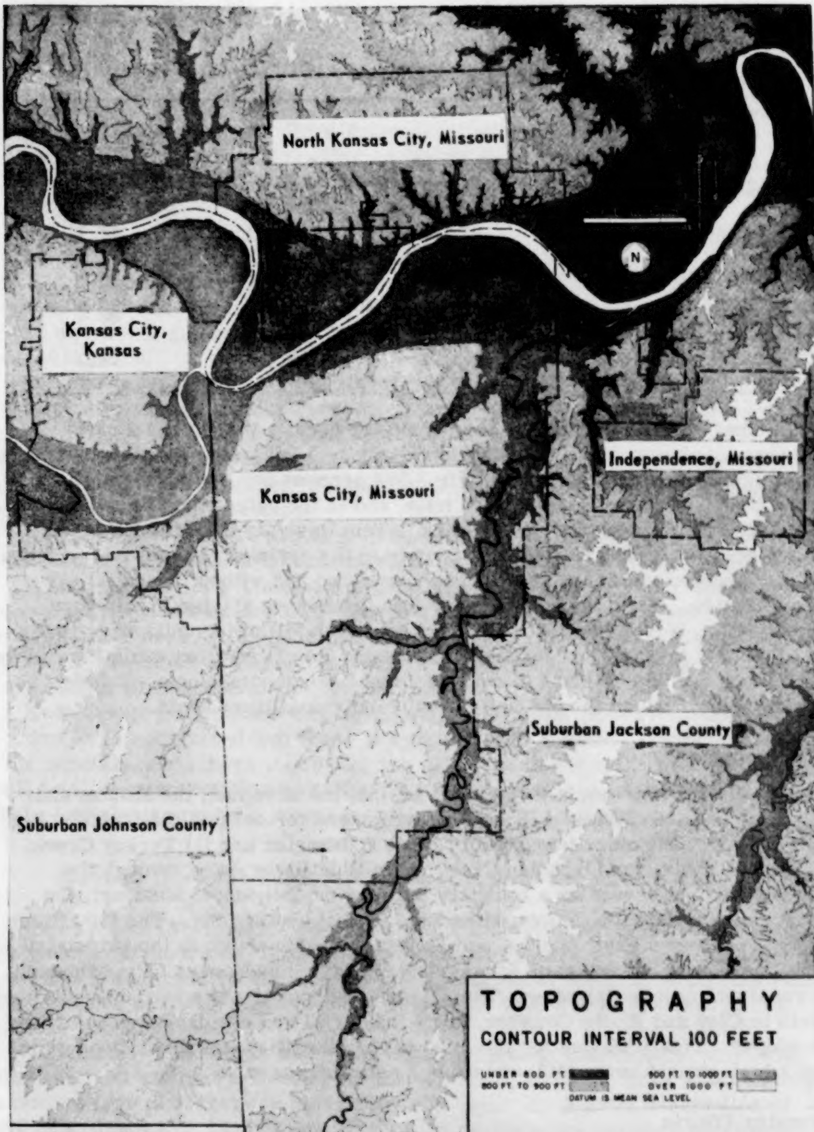


Fig. 1. Water courses from many suburban areas flow through Kansas City.

city planning commissions in both Kansas City, Missouri, and Kansas City, Kansas. Jackson County, Missouri and Wyandotte County, Kansas also have planning commissions. The jurisdiction of each of these planning groups is limited. The only area in which planning is being done on a truly metropolitan basis is in the field of highways and trafficways. The credit for the progress in this field must be given largely to federal and state highway engineers and officials who have done an effective job of educating the public to the need and advantages of metropolitan planning of traffic facilities. Similar benefits could be realized from metropolitan planning of sewerage facilities. Zoning and subdivision regulations vary widely in the several political subdivisions with each particular area attempting to attract new residential, commercial, and industrial growth into its area by making its requirements a little less strict.

While there is widespread agreement among technical and professional personnel employed by the various governmental units relative to the need for, and desirability of solving area-wide problems, one of which is sewerage, on a metropolitan basis, they have not yet been able to secure any expression of willingness on the part of responsible local government officials to surrender any of their prerogatives or controls in these matters to a metropolitan agency. None of the planning being done by the various planning commissions includes any long-range planning for sewers. The mayor of Kansas City, Missouri has recently organized a metropolitan advisory council on sewerage. It is hoped that this group will be successful in obtaining essential bi-state legislation to permit bi-state metropolitan planning and operation of sewerage systems.

Enabling Sewer Legislation—Missouri

Legislation was enacted in Missouri in 1941 (Missouri Laws of 1941, p. 557) which authorized the governing bodies of certain counties to organize sewer districts either upon receipt of a petition or upon their own motion and to employ engineers to prepare plans and to construct sewerage systems using (1) general obligation bonds, (2) revenue bonds, (3) any funds which they may have on hand for this purpose, and (4) state or federal grants to finance the cost of such facilities. The law also made provision whereby a private developer could request the organization of a sewer district including his subdivision, and the sewers constructed by the developer could be turned over to the County for operation. According to reports of Missouri Health Department officials, this procedure was used exclusively in Jackson County prior to 1957. No county general obligation bonds or revenue bonds had been used to construct sewers in Jackson County, Missouri, prior to 1957. The Missouri Constitution requires a 2/3 majority for approval of general obligation bonds and 4/7 majority for revenue bonds. Another interesting practice in Missouri is the use of tax bills for payment of the contractor. County tax bills generally run for a period of 5 years and bear 8% interest. Property owners are given 30 days to pay off their tax bills after which they pay them in 5 annual installments plus the 8% interest. Contractors who need operating cash can, of course, sell the bonds to various investors at a discount. All of these factors doubtless explain why developers desire to construct their own sewers.

It is reported that there are some 40 small sewer districts in Jackson County each with its own separate little sewage treatment plant which is operated by the County. Funds for operation of these small plants are obtained from a small mill tax levied against all the property by the county.

Enabling Sewer Legislation—Kansas

The first legislation was enacted in 1927 (Chapter 165, Session Laws, 1927) and authorized the county commissioners in certain counties, upon receipt of a petition to organize sewer districts, to employ engineers to prepare plans for sewering the districts, to issue bonds, and to let contracts for the construction of sewers in accordance with approved plans. Cost of the construction is to be met by special assessments against all the land in the district. Under this particular statute the credit of the entire county is pledged to support these bonds. The sewer systems built under this law have been quite limited in area, generally a few subdivisions.

In 1945, legislation was enacted (Chapter 179, Session Laws, 1945) authorizing the County Commissioners of certain counties to create Main Sewer Districts embracing all or a major portion of the land in a watershed. The Commissioners could create the main district on receipt of a petition signed by the owners of 10% of the land area. Engineers could be employed to make a preliminary feasibility study and cost estimate. Upon receipt of a favorable engineering report, a bond election to authorize the issuance of bonds is held. The amount of bonds that can be issued is limited to 20% of the assessed value of the property of the district including improvements. Under this statute only the property of the district is pledged to secure these bonds. This legislation is used in Johnson County, Kansas, where some 20,000 acres of land is being sewered by three main sewer districts, all under one governing body, the Johnson County Commissioners. Land within the corporate limits of cities may be included in the district if city officials consent.

In 1941, legislation was enacted in Kansas (Chapter 399, Session Laws, 1941) authorizing certain township boards to (1) create sewer districts upon receipt of a petition or on their own motion, (2) employ engineers, (3) let contracts for construction of sewers, (4) issue general obligation bonds to pay the cost of such sewers, and (5) provide for payment of the bonds by special assessments against property in the district. Under this law only the property of the district is pledged to pay the bonds, but no limit is placed upon the amount of bonds that can be issued. This legislation is used widely in Wyandotte County. No provision was made for inclusion of incorporated areas.

Public Sewer Systems

Today, practically all new subdivisions in the metropolitan area are being provided with sewers. In Johnson County, Kansas, the sewer systems operated by the three main sewer districts and a private company will eventually serve an area of approximately 20,000 acres in the Indian Creek, Brush Creek, and Turkey Creek watersheds.

In Wyandotte County sewers are being provided by 18 or 19 small township sewer districts organized by township authorities. Several of these systems are connected to the Kansas City, Kansas, system. Others have their own treatment plants.

In Jackson County, Missouri, developers are petitioning the county authorities to organize their subdivisions into sewer districts. Here the developer constructs the sewers and includes the cost in the price of the house. These small plants, generally Imhoff tank-open sand filter plants or lagoons, are operated by the county. In both states FHA and VA have given strong support to these sewer programs. Both agencies prefer to have the builder provide water and sewer utilities and include the cost in the overall price of the house.

Efforts in Developing Interstate and Intercounty Systems

Late in 1953 the Jackson County Court employed consulting engineers to make a study of sewers for the Dykes Branch-Indian Creek drainage area. This report has been completed and is being studied by the Jackson County Court. It recommended the construction of a comprehensive integrated sewer system to serve the entire Indian Creek drainage area, a large part of which lies in Kansas with a treatment plant located at the junction of Indian Creek and the Blue River as the most desirable solution.

Concurrently with this Missouri study, the Johnson County Commissioners in Kansas engaged engineers to study ways and means of sewering the Indian Creek drainage area in Kansas. This report took cognizance of the desirability of locating the treatment plant at the junction of the Blue River in Missouri, but the legal and financial difficulties inherent in such a plan caused the engineering firm to recommend the construction of a treatment plant in Kansas on Indian Creek.

Similar possibilities for joint action between Johnson and Wyandotte counties exist in the Turkey Creek watershed which drains into the heart of Kansas City, Missouri. The general topography is such that it is feasible from an engineering standpoint to bring all the sewage from the Turkey Creek watershed to the Kaw River by gravity for treatment. Inter-community political and commercial rivalry again presents such formidable obstacles to this approach that no serious public consideration has been given to it to date. Instead numerous small sewage treatment plants have been constructed along the upper reaches of Turkey Creek and discharge treated sewage into the otherwise intermittent flowing stream.

A similar impasse developed on Brush Creek in 1945 when it was proposed that sewers in the Brush Creek watershed in Kansas be extended through Kansas City, Missouri, and connected to the city system at the Blue River. After many months of conferences with Kansas City, Missouri, officials the Johnson County officials gave up, built a pump station, and now pump several million gallons of sewage each day over a ridge to a treatment plant at the edge of a residential area. Similar examples of lack of inter-community cooperation could be cited in the Missouri area, but these suffice to indicate clearly the difficulties engineers and others have encountered in attempting to solve some of the sewerage problems of this area in the most logical and economical manner.

Financing

Sewerage facilities in this metropolitan area are being financed in a variety of ways. The incorporated areas use general obligation bonds, special assessment bonds, and revenue bonds. In the unincorporated areas of Clay and Jackson Counties all sewer construction has been financed by the developers. In Johnson and Wyandotte counties in Kansas, general obligation bonds and private capital of developers have been used.

None of the public financing methods are well adapted to the situation existing in the outer urban fringes. Lack of adequate assessed valuation in these areas coupled with a high degree of speculation makes use of any of the conventional methods of public financing almost impossible. The practice of requiring the developer to finance the cost of sewerage tends to limit development to large operators.

Investment brokers have suggested that several remedial measures be taken to alleviate the situation:

(1) Have counties, or cities, underwrite the sewer district bonds as is now being done in Washington Suburban Sanitary District (Kansas statutes permit this procedure but it has not been used widely).
(2) Merge water and sewage utilities and pledge the income from water sales and sewer service charges to pay revenue bonds used to construct both water and sewerage facilities.
(3) Create county or state revolving funds that could advance loans to these rapidly developing areas until such time as bonds could be sold through conventional financing channels at which time the revolving fund would be reimbursed.

A unique financing procedure developed in Wyandotte County has proved successful so far. Briefly, the procedure consists of (1) organizing the sewer district on a small scale, generally a few hundred acres of land owned by a large developer or group of developers, (2) voting a large amount of general obligation bonds, (3) issuing sufficient bonds to build a sewer system for the entire watershed, and (4) providing for the payment of the bonds from tapping fees to be collected from areas outside the district and from assessments against the property within the district. The amount of the tapping fee is determined by dividing the total amount of bonds by the number of houses it is estimated will be built in the watershed when fully developed. For example, if $2,000,000 in bonds were to pay for the sewerage system and it is estimated that there will be 10,000 homes in the watershed when fully developed, each house located outside the sewer district would pay $200 tapping fee. Wherever possible a "call" feature is inserted in the bonds and in several instances the bonds have been retired long before maturity. In other cases it was necessary to eliminate the call feature in order to sell the bonds at a proper rate.

Operation and Maintenance

All of the public sewer district legislation applicable to the fringe area of Kansas City provides that the county or township governing bodies shall also be the governing body of the sewer districts. This procedure has both its good and bad points.

The principal advantage is that a single board is made responsible for all the districts in a given area. This facilitates the consolidation, cooperation, and joint use of facilities. In this regard the county governing bodies are superior to the township bodies as a general rule. The pattern that is followed in any given community is in large measure determined by the relative political strength of the two levels of government. In Wyandotte County, Kansas, where the township pattern is used, the townships are strong elements of local government. In Missouri the county is the basic part of local government.

The principal disadvantages of using the existing county and township officials as governing bodies for sewer districts are: (1) these officials are already overloaded with other responsibilities and may be inclined to neglect the problems of the sewer district, (2) these officials are in many instances political leaders, who may be inclined to make the sewerage service a political football, and (3) these county and township officials are elected primarily for other purposes and may know very little about operation of sewerage systems.

In spite of these shortcomings, the use of existing county and township boards may be preferable to the election of a new and separate board for each district. If a non-political independent board is to be provided, it would seem desirable that it be given jurisdiction over an entire drainage basin or group of drainage basins.

Role of Health Agencies

In both Missouri and Kansas the state and local health authorities have strongly discouraged the use of individual septic tank systems in the urban fringes of Kansas City. The engineering personnel in these departments have spent many hours in conference with builders, consulting engineers, local officials, and legislators pointing out the shortcomings of these systems and the need for enabling state legislation that will permit local unincorporated areas to create sewer districts.

Role of the Central Cities

Kansas City, Kansas and Kansas City, Missouri have followed widely different courses in regard to annexation. Kansas City, Missouri with its city manager form of government has always encouraged annexation as protection against encirclement such as occurred at St. Louis. In the 14 years between 1940-1954, Kansas City, Missouri expanded its area from 60 square miles to 82 square miles. Kansas City, Kansas, on the other hand, has not expanded its area appreciably since 1925.

Both cities sell water to outlying areas through the Kansas City Suburban Water Company, a privately owned water utility, and through township water districts. Electric power in the area is distributed by a private power utility and by a municipally owned utility in Kansas City, Kansas operated by an independent board. Electric power is available over the entire metropolitan area from one or the other of these sources. Neither city has shown particular interest in extending sewer service to outlying areas. This doubtless stems in part from the fact that (1) their present municipal systems are in many instances inadequate to serve the city properties, and (2) debt limitations coupled with general public opposition to additional tax burdens make it politically difficult to propose expansion of the present system to serve any outlying areas. None of the sewage from either of the city systems is treated, but is discharged into the Kansas and Missouri Rivers without treatment. The practice, of course, will soon be terminated. The Kansas River Basin Policy requires Kansas City, Kansas to cease discharge of raw sewage into the Kansas River by 1953.

CONCLUSION

The following points apply to the sewerage problem as it exists in the Kansas City metropolitan area:

(1) The provision of adequate sewerage services in the Kansas City suburban areas will require determination and persistance on the part of all persons interested in the problem.

(2) The problem of developing sewers and sewage treatment facilities on a regional basis has not received the support from civic leaders in the greater Kansas City area that it must have if the needs of the area are

to be met in an effective and economical manner. (While many of these leaders admit privately that present practices are cumbersome and inefficient, local city officials are reluctant to surrender any local control to a metropolitan agency.)

(3) The efficient and orderly provision of adequate sewerage services for the entire area can best be solved by a metropolitan authority similar to that developed by Toronto Metropolitan District and the Chicago Sanitary District.

(4) There is an urgent need for a comprehensive engineering and economic study to show comparative costs of handling sewage under the present practices and under a metropolitan authority. Getting money to finance such a study is the first problem.

Appendix E.—Interim Treatment Plants*

In the Report of the Committee and the appendices describing the efforts in the various metropolitan areas repeated reference was made to the urgent need for the development of better ways for implementing the master plan and greater use of facilities to permit the master plan to be accomplished in a reasonable time.

To fully meet the need, solutions must be found to two widely different problems. They are: (1) How to keep capital investments in interim treatment plants located on temporary sites at a minimum so that they can be abandoned in a few years without great sacrifice and (2) designing the initial treatment facilities which are constructed on permanent sites so that they can be integrated into the final plant.

The suggested solutions appearing in this appendix are not intended to be a substitute for engineering judgment or a listing of all possible solutions. Each solution has its limitations which the engineer should keep in mind in making his final selection.

First of all, these suggested solutions are interim measures designed to meet the need for a relatively short period while the area is developing an adequate financial base to finance the construction of the permanent facilities.

Secondly, the use of interim plants should be limited to areas where (1) a suitable administrative mechanism such as a sewer district, is available for planning, constructing, enlarging, consolidating, and operating both interim and permanent treatment facilities, and (2) a master sewer plan for the drainage area has been prepared and officially adopted.

With these general limitations in mind, the remainder of this discussion will be devoted to a discussion of the general types of plants that have proven successful for interim treatment of sewage in suburban areas.

Septic Tank-Sand Filter

This type of plant is best suited for use at small developments of 50 houses or less. If properly operated, effluent from such plants will be clear and odorless and can be discharged into intermittent flowing streams without nuisance. Provision must be made for periodic removal and disposal of sludge from the septic tank.

---

*The Committee thanks Mr. Carl Hodgkinson and Mr. Ivan F. Shull of the Kansas State Board of Health for the preparation of this section.

These plants are likely to produce some offensive odors for short periods whenever the filter is dosed. If ponding of sewage on the filter is permitted to occur, offensive odors will persist until the filters are cleaned.

For these reasons septic tank-sand filter plants should be located at least 500 feet from existing houses and new houses. This type of plant requires regular care and should not be considered unless adequate maintenance by a qualified operator can be provided.

The following data* based on research at the University of Florida shows the B.O.D. removals for various types of sand filters at various rates of loading.

Stabilization Ponds

For developments with 50-200 houses, waste stabilization ponds have considerable merit particularly if the interim plant is to be located on a temporary site that must be abandoned later. The following general rules should be observed in locating and designing stabilization ponds:

1. Maintain at least 500 feet separation from residences.
2. Keep the total first stage B.O.D. loading below 75#/acre/day.
3. Provide means for varying the water depth between 18 and 60 inches.
4. Enclose the area with a woven fence.
5. Locate the inlets near the center and the outlets at points farthest from inlets.
6. For more than 300 persons, provide at least two cells that can be operated in series or in parallel.
7. Seal the bottom and sides with compacted soil and soil additives when necessary to reduce seepage. This is a major factor in areas of medium to high evaporation.
8. Provide an auxiliary supply of water to maintain minimum water levels during periods of drought if necessary.

Imhoff Tank-Trickling Filter

This type of plant is best suited for use on a permanent site. Construction costs tend to be higher in most instances than for other type plants. Imhoff tanks are malodorous at times and therefore should be well removed from residential areas. For maximum economy of construction the design should permit a maximum use of low cost local materials that require a minimum of skilled labor. The transfer of raw or partially digested sludge to other nearby plants for digestion will reduce sludge storage requirements and decrease the costs where facilities are available.

While Imhoff-trickling filter plants can withstand a certain amount of abuse load wise, they must be given routine operational attention if odor nuisances are to be avoided.

Larger Developments

For developments of 50-500 houses, any of the following types of treatment plants are suitable for interim use:

*Intermittent Sand Filter Loadings. Furman, Calaway and Grantham. Sewage and Industrial Wastes, Vol. 27, p. 261, March 1955.

Test Data for Filter Units

	Loading (1000 gpad)	No. of Samples	Infl. (ppm)	Infl. (lb./a./day)	Effl. (ppm)	Removal (%)
Filter Unit No. 8 Sand Size-0.46 mm; Unif.Coef.-2.79; Bed Depth- 30 in.	125	7	142	148	13	90.8
	150	29	174	217	24	86.2
	175	8	143	208	28	80.4
	200	35	96	160	17	82.3
	225	14	106	198	15	85.8
	250	18	87	181	15	82.8
	300	36	121	299	19	84.3
	325	10	152	412	14	90.8
Filter Unit No. 10 Sand Size-0.25 mm; Unif. Coef.-2.24; Bed depth- 30. in.	100	7	142	118	2	98.6
	150	17	161	201	3	98.1
	175	8	143	208	3	97.9
	200	35	96	160	3	96.9
	225	14	106	198	2	98.1
	250	16	85	177	2	97.6
	300	34	122	302	3	97.5
Filter Unit No. 12 Sand Size- 0.31 mm; Unif. Coef.- 3.26 Bed Depth- 30 in.	100	9	136	113	3	97.8
	150	17	160	200	4	97.5
	175	7	143	208	4	97.2
	200	35	96	160	5	94.8
	225	14	106	198	4	96.2
	250	15	84	175	4	95.2
	300	32	133	332	7	94.7

1. Waste stabilization ponds.
2. Primary treatment plus secondary treatment with trickling filter.
3. A combination.

For developments of over 500 houses, serious consideration should be given to phase or step construction on a permanent site. If the cost of extending a main sewer to the permanent plant site is prohibitive, the possible use of a temporary pump station and a steel force main laid on top of the ground should be explored.

On a permanent site the first stage of construction might consist of a stabilization pond. As the load on the pond approaches its upper limit, an Imhoff tank designed for future conversion to a sludge digester could be provided as a second step. The third step might be the addition of a high rate trickling filter and final settling basin with the stabilization pond being used for final polishing of the effluent. A fourth step could include the construction of a primary clarifier and the conversion of the Imhoff tank into a digester. By the time the capacity of these facilities are reached, enlargement of the plant to its ultimate capacity by constructing additional clarifiers, digesters, filters and sludge drying facilities as are needed should be feasible.

To successfully use stage or phase construction, the governing body must be in position to undertake the necessary enlargements as they are needed. This can be accomplished by (1) voting the full amount of bonds to be issued only as needed or (2) by empowering the governing body to issue bonds for enlarging the plant after initial approval of the entire project by the voters.

The following schematic sketches show how the various suggested practices would apply to a community having 100 houses initially with a predicted ultimate development of 2,400 houses:

First Stage—includes temporary pump station (if required) with the piping and pumps designed to serve an ultimate of 400 houses and the first cell of waste stabilization pond designed to serve 100 houses. Distribution box should be stubbed out for second cell.

Second Stage—includes second cell, inlet line from the distribution box and effluent box.

Third Stage—includes the third cell to receive effluent from first 2 cells.

Fourth Stage—includes Imhoff tank, which is designed primarily as digester and the first sludge drying beds.

Fifth Stage—includes new permanent pump station and sludge control house distribution box stubbed out for an ultimate of four clarifiers, first primary and final clarifiers, more sludge drying beds, and a high rate filter with a temporary line to the waste stabilization pond for final treatment. The Imhoff tank is converted to a digester with the Imhoff sludge salvaged for seed.

Sixth Stage—includes an additional primary and final clarifier, second digester, second filter and more drying beds.

Seventh Stage—waste stabilization ponds are abandoned and the area used for a new filter and more sludge drying beds. The filter is designed to provide sufficient volume with the existing filters, to operate at standard rate. Additional primary and final clarifiers together with sludge heater are provided.

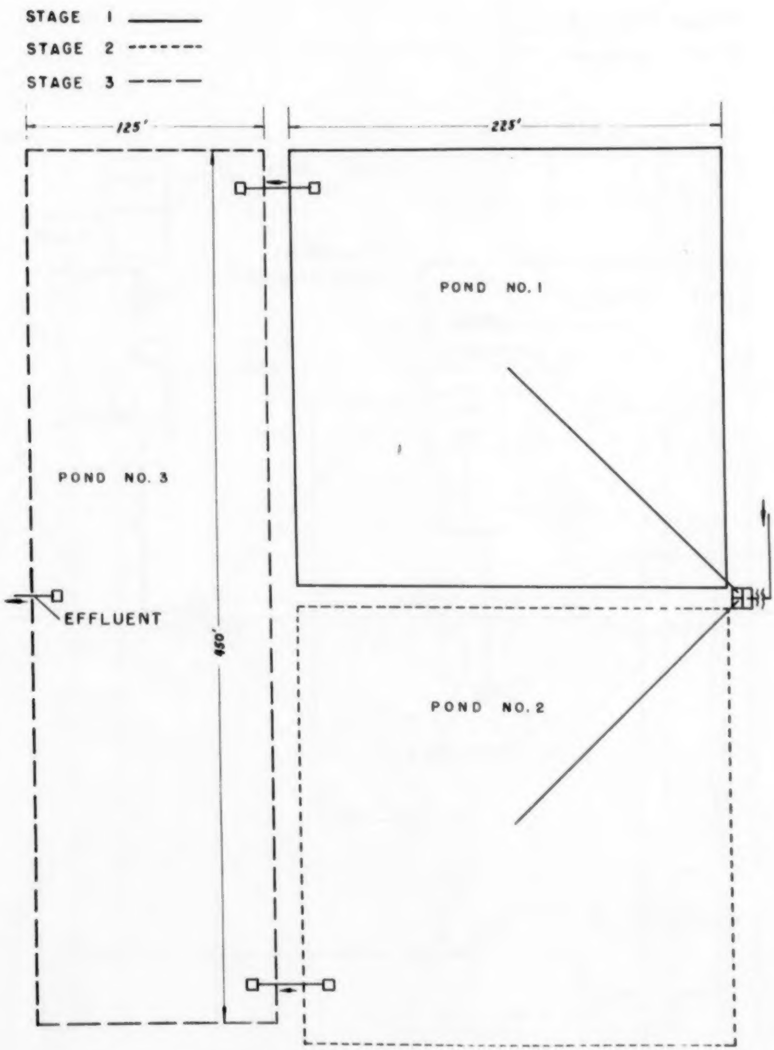
Eighth Stage—includes additional primary and final clarifier and more drying beds. The filter is made adequate by recirculation.

Plates 1-4 are to indicate how the procedure could work. They are intended to be illustrative of step development, and they include more stages than will generally be necessary.

Usually it will be economically feasible to eliminate one or more of the stages, or if the area permits, the waste stabilization pond cells might be increased in size. This would allow the construction of larger clarifier units with a subsequent reduction in the number of filters and clarifiers. Larger sizes in the permanent plant construction or elimination of the Imhoff tank stage might also be accomplished by providing the fourth cell of the waste stabilization pond. The provision of a fourth cell in the waste stabilization pond might also permit some reduction in size of permanent facilities.

Respectfully submitted,

B. B. Berger	W. T. Ballard
Emil C. Jensen	James B. Coulter
Charles L. Senn	E. L. Ruppert
W. W. Towne	Ivan F. Shull
Dwight F. Metzler, Chairman	George W. Reid, Chairman
Committee on Public Health Activities of the Sanitary Engineering Division	Task Force

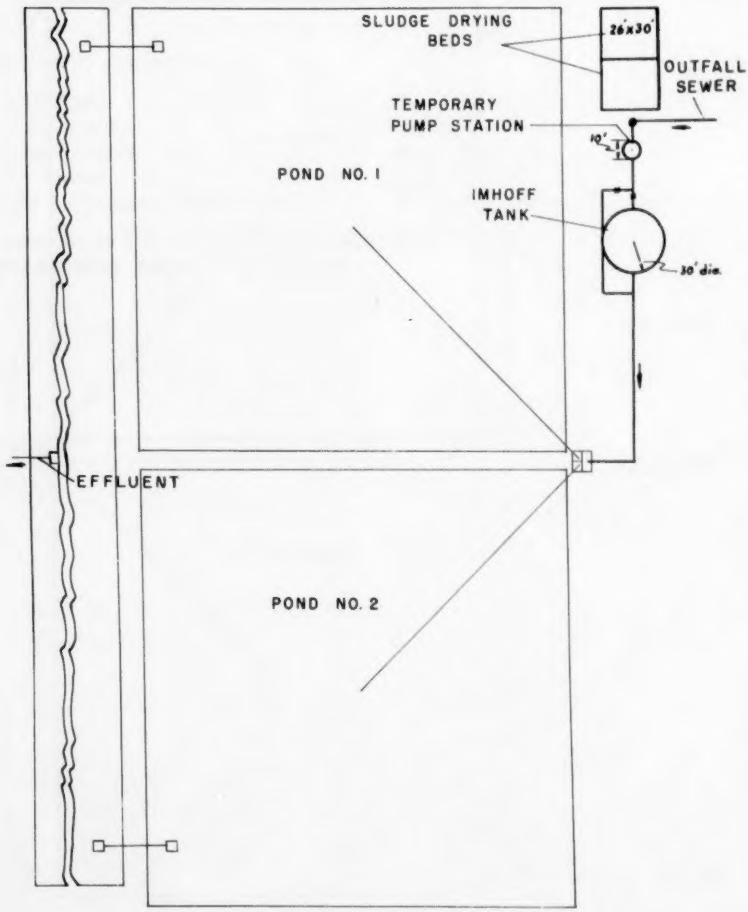


STAGE CONSTRUCTION OF SEWAGE TREATMENT FACILITIES

PLATE 1 WASTE STABILIZATION PONDS

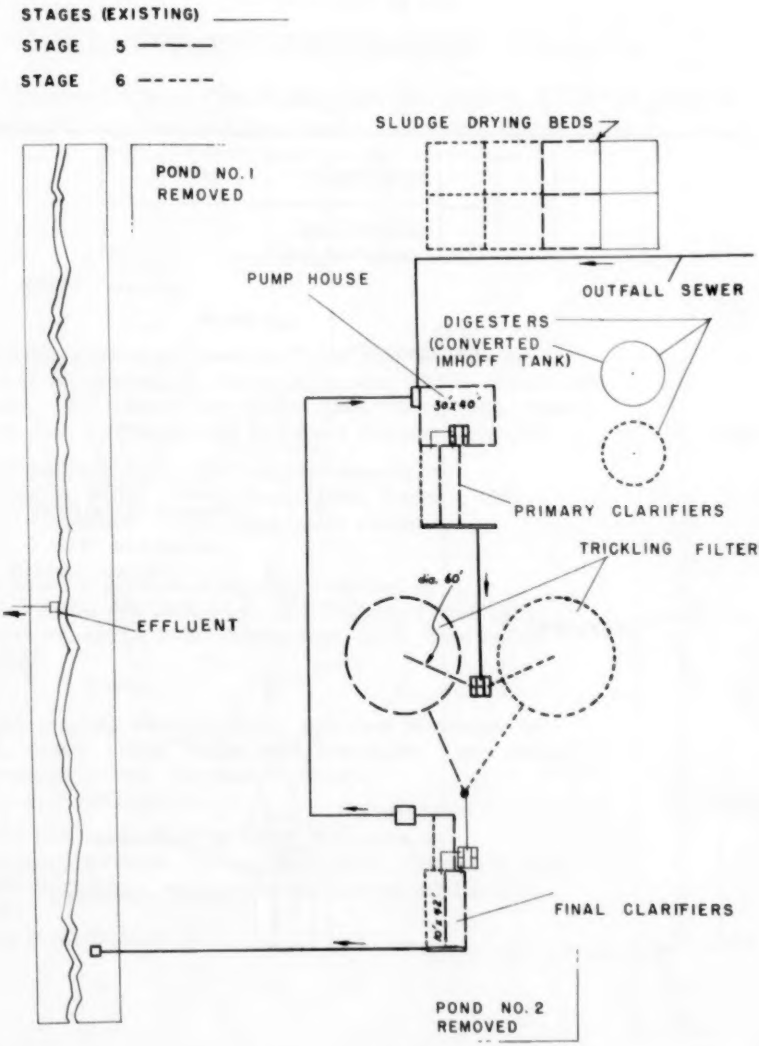
STAGES (EXISTING) ———

STAGE 4 ———



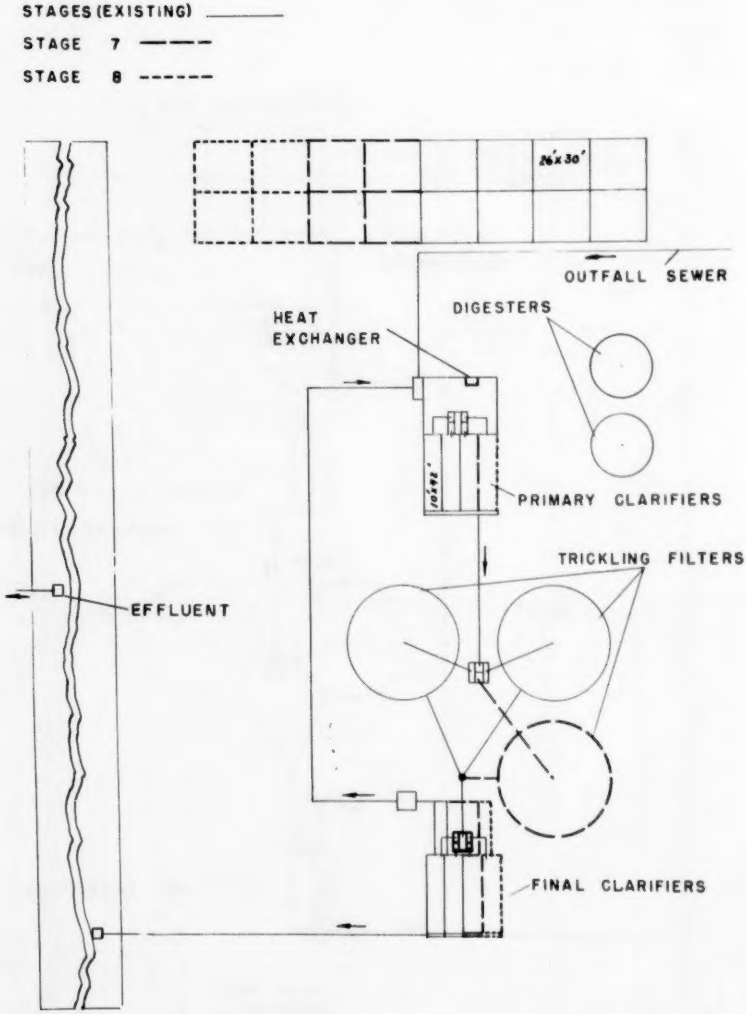
STAGE CONSTRUCTION OF SEWAGE TREATMENT FACILITIES

PLATE 2 PRIMARY TREATMENT ADDED



STAGE CONSTRUCTION OF SEWAGE TREATMENT FACILITIES

PLATE 3 CONVERSION TO TRICKLING FILTERS



STAGE CONSTRUCTION OF SEWAGE TREATMENT FACILITIES

PLATE 4 COMPLETED PLANT

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1688)

[illegible]

TRICKLING FILTERS SUCCESSFULLY TREAT MILK WASTES

Closure by Paul E. Morgan and E. Robert Baumann.

PAUL E. MORGAN[1] and E. ROBERT BAUMANN,[2] A. M. ASCE.—The writers would like to state that the aeration mentioned in the discussion was not in use during all of the surveys. This unit was not installed until after the first series of sampling and no increased efficiency could be detected in the plant operation. The writers wish to thank Mr. Chase for his discussion and for stating the results of his survey on the same milk waste treatment plant. They seem to agree very well with the original study.

1. Asst. Prof., Civ. Eng. Iowa State College, Ames, Iowa.
2. Associate Prof., Civ. Eng., Iowa State College, Ames, Iowa.

DISPERSAL OF POLLUTION BY TIDAL MOVEMENTS[a]

Discussion by Alex N. Diachishin

ALEX N. DIACHISHIN,[1] A. M. ASCE.—Mr. Niles is to be commended for his realization of the marked differences that exist between a tidal, brackish stream and its fresh-water counterpart. Not only is there the physical appearance of pollutional material upstream of a pollution source as Mr. Niles stresses but there are subtler differences evident in brackish streams. It should be stressed that a stream can be either tidal, brackish, or both. For instance the Hudson River at Albany is definitely tidal, with a tidal range about equal to that at the mouth of the Hudson some 150 miles to the south, however, at Albany the Hudson is not brackish. Therefore, at Albany we expect a distribution of pollution both upstream and downstream of a pollution source, a change in reaeration rate from a non-tidal stream—due to the velocity of the tidal current, but a bacterial death rate equivalent to that of a fresh-water stream. However, at the mouth of the Hudson in New York Harbor, the Hudson River is both brackish and tidal. Therefore, in addition to all of the physical changes engendered by the tidal effect we also expect a change in the biological regimen of the stream occasioned by the change in biota. It is in a brackish stream that we expect the markedly different bacterial death rate for coliform organisms as that has been shown by other investigators.(1)

Even with rapid bacterial death rates tidal currents are so swift that a pollution source can have a considerable radius of influence many times greater than if the velocity were governed by fresh-water flow, as is noted by Mr. Niles. In the Upper East River of New York Harbor, for instance, there is virtually no fresh-water flow into the River during the summer months, the East River merely providing a linkage between Long Island Sound and the Upper Bay of New York Harbor. In a fresh water "lake" of this size there would be a virtual stagnation of wastes, the only circulation being that induced by wind and weather. However, in actuality the tides are such that maximum velocities of 1.5 to 2.0 knots are not uncommon and tidal travels of five to eight miles are experienced during one-half of a tidal cycle. There is a reversal of the tidal current in the East River with approximately the same velocities so that there is little net transport of the water itself. However, there is considerable mixing and not all of the pollution returns to the site where it is discharged.

Mr. Niles indicates that the tidal velocities that are encountered in a tidal stream can be estimated on the basis of the volume required to fill the tidal prism, the tidal prism being defined as that volume of water between the high and low tide water levels.

a. Proc. Paper 1408, October, 1957, by Thomas M. Niles.

1. Cons. Engr., Bergenfield, N. J.

This implies that there is a more or less simultaneous occurrence of high and low water throughout the length of the estuary, which does not appear to be true for some estuaries. For instance, on the Delaware River, which is used by Mr. Niles as an example, the time of high or low water is approximately two hours later at Trenton as compared to Philadelphia.(2) Moreover, the use of a tidal prism implies that the minimum current occurs at the time of high or low water, which is definitely not the case for the Delaware River throughout much of its lower length.

In view of these considerations it appears advisable to employ tidal velocities that have been measured rather than calculated. In those instances where current data is not available the measurement of currents with meters or other appropriate devices is indicated.

Mr. Niles makes note of the fact that in a tidal stream there is a dumbbell-shaped slug of pollution that oscillates to and fro about the center of pollution as the tides alternately flood and recede. It is well to point out in this regard that the average concentration is a maximum at the focus of pollution as can be easily shown. However, the area that experiences the greatest range of pollution concentrations is that which lies just on the edge of the circle of influence of the pollution. Of course, all of the preceding is made without regard to the influence of mixing. Mixing in a tidal stream is such as to further widen the circle of influence of pollution so as to make the concentration of pollution lower in highly polluted areas and higher in areas that are relatively less polluted. This in effect stretches out the dumbbell-shaped curve to two ellipses joined end to end.

In many tidal and brackish streams a knowledge of pollution parameters other than the dissolved oxygen concentration is often desirable. This is, of course, the case for shell-fish bed areas and for beach areas that are used for swimming. In tidal streams such as these there is no assurance that a reduction of dry-weather pollution at the source will result in a proportionate decrease in the bacteriological concentration of coliform organisms at some upstream or downstream site.

Moreover, even in tidal streams where oxygen concentrations are of primary concern, the application of a simple proportion proposed by Mr. Niles, such as that shown below, should be applied with care.

$$\text{Allowable BOD Load} = \text{Desired D.O. Deficit}\ \frac{\text{Existing BOD Load}}{\text{Existing DO Deficit}}$$

It is entirely possible that in areas where minimum flows occur during the period of maximum temperatures such as is the case for much of the United States—that algal activity is at a maximum during this hot, low flow, period. The capacity of algae to add considerable amounts of oxygen to lagoons has already been noted(3) and the ability of algae to add considerable amounts of oxygen to streams as well is a reasonable assumption. Thus, even in fresh-water streams, it appears that the period of minimum flow does not necessarily coincide with the period of minimum dissolved oxygen even after temperature effects have been taken into account. This may account for some of the lack of correlation of dissolved oxygen with river flow as noted by Mr. Niles, although it would have been instructive if the plots of dissolved oxygen versus flow shown by Mr. Niles would have also included the temperatures associated with the data.

However, even after taking temperatures into account the mixing provided in tidal streams is an important factor in self purification which may

completely override the diluting effect of fresh water flows when fresh water flows are comparatively small.

Many sanitary engineers are not familiar with the marked departures of fresh-water streams from tidal and brackish streams and Mr. Niles has provided valuable service in noting the extent of this departure. However, tidal estuaries are complex hydraulic and biological mechanisms and it would appear that simplified mathematical models—simplified even beyond their fresh-water counterparts—are not universally applicable to all estuaries.

REFERENCES

1. Vaccaro, Ralph T. et al., "Visibility of Escherichia Coli in Sea Water." A.J.P.H. 40:1257 (Oct., 1950).

2. United States Coast & Geodetic Survey "Tide Tables" East Coast—1957, p. 209.

3. Towne, W. W., Bartsch, A. F. and Davis, W. H., "Raw Sewage Stabilization Ponds in the Dakotas," Sewage and Industrial Wastes 29: 377 (April 1957).

VORTEX FLOW THROUGH HORIZONTAL ORIFICES[a]

Discussion by C. J. Posey

C. J. POSEY,[1] M. ASCE.—For the case of purely radial inflow, the experimental results obtained by the authors corroborate the results of the Iowa tests.(4) The authors state that "The vortexes which formed during this stage of the testing, with uniform approach conditions, were small and unstable. The direction of rotation would change from clockwise to counter-clockwise with no definite pattern and the air core would form only momentarily. These vortexes were found to have no noticeable effect on the standard orifice coefficients." The corresponding conclusion from the Iowa tests was that "With purely radial inflow the vortex is small and transitory; its effect on discharge is negligible." It was further determined in the Iowa tests that if "the tangential component of inflow is shut off after the vortex has become stable, it will soon die out." The problem of vortex formation when the inflow is purely radial would therefore seem to be pretty well settled, with the possible exception of the case of a tank so large and perfectly symmetrical that the earth's rotation would have a measurable effect.(9)

In the Iowa investigation the values of C in the standard orifice formula, Eq. (21), were computed for those tests in which the inflow was purely radial, and found to range from 0.562 to 0.582, based on the depth as measured by piezometer holes 13 inches from the center of the 4-inch diameter sharp-edged orifice.(10) Figs. 2 to 6 seem to indicate that the authors considered that C ought to be 0.600 or slightly less. No values are given nor plotted points shown, however, for vortex numbers less than 0.3.

Comparison of the quantitative results of the two investigations for cases in which the inflow was partially tangential is made difficult or impossible by the authors' unfortunate choice of variable. Apparently it is necessary to determine the water surface profile measurements and use the theoretical equation for the water surface hyperbola to determine the "vortex number" in terms of which the results are summarized. The designer who wishes to use the authors' data to determine the effect of a vortex upon the discharge through a certain outlet will be hard put to estimate the value of the "vortex number." The authors themselves give no estimation of its value for the sewerage vortex chambers described in the last part of the paper.

The variable used in summarizing the results of the Iowa tests, on the other hand, can be easily computed if the angularity of the approach flow is known. It is the ratio of tangential component of the average velocity to its radial component, or the tangent of the angle θ between the resultant velocity

---

a. Proc. Paper 1461, December 1957, by J. C. Stevens and Richard C. Kolf.

1. Prof. and Head, Dept. of Civ. Eng., State Univ. of Iowa, Iowa City, and Director, Rocky Mountain Hydr. Lab., Allenspark, Colo.

and the radius from the center of the outlet. The radial component of the average velocity at any point is the discharge divided by the surface area of a hypothetical cylinder through the point, concentric with the outlet. (The cylinder is taken to have large enough diameter that the depth is nearly equal to that at a considerable distance from the outlet.) The tangential component of the velocity through the sides of this cylinder depends upon the strength of the vortex, or circulation. It can be computed as the sum of the product of the discharge, tangential component of velocity, and radius for each source of inflow, divided by the product of the total discharge and the radius of the cylinder. The ratio of tangential to radial components of velocity, $\tan\theta$, is constant for any radial distance from the center of the orifice if no appreciable tangential force or momentum is introduced. This corresponds to the authors' statement that "the streamlines are equiangular and are theoretical logarithmic spirals." Tan θ can be shown to be equal to the ratio between the circulation and the discharge per unit depth, and is dimensionless, as is the authors' "vortex number."(10)

Fig. A, summarizing the results of the Iowa tests, is reproduced for the benefit of designers who may wish to estimate the effect of non-radial approach on the discharge through a bottom outlet. The validity of $\tan\theta$ as measure of effect of the vortex is apparent, especially when it is realized that in these tests both the radius and the velocity of the tangential inflow were varied over a wide range.

The following practical example illustrates the use of Fig. A. A tank 10 feet in diameter has a 6-inch sharp-edged circular orifice in the center of its bottom, which is horizontal. Pipe connections to the tank introduce tangential discharges of 0.20 and 0.15 cfs through two 3-inch pipes at distances of 4 and 5 feet, respectively, from the center. A 2-inch pipe introduces 0.05 cfs at the edge of the tank, inclined 45° with the radius and opposing the rotation caused by the two 3-inch pipe inlets. Another 0.6 cfs enters the tank radially. It is required to determine the depth of water at the outside edge of the tank.

For computational purposes, assume a hypothetical cylinder 4 feet in diameter and H feet high. The components of the average velocity through the sides of the cylinder are computed as follows:

$$Q_1 \cdot V_1 \cdot R_1 = 0.20 \times \frac{0.20}{.049} \times 4 = 3.27$$

$$Q_2 \cdot V_2 \cdot R_2 = 0.15 \times \frac{0.15}{.049} \times 5 = \underline{2.30}$$

$$5.57$$

$$Q_3 \cdot V_3 \cdot R_3 = 0.05 \times \frac{0.05}{.022} \times .707 \times 5 = \underline{0.40}$$

$$5.17$$

$$\text{Tangential Component} = \frac{\Sigma QVR}{R\Sigma Q} = \frac{5.17}{2 \times 1} = 2.58$$

$$\text{Radial Component} = \frac{\Sigma Q}{H\pi D} = \frac{1.0}{H\pi \cdot 4} = \frac{0.079}{H}$$

Hence

$$\tan\theta = \frac{2.58H}{0.079} = 32.6H$$

Assuming that H = 3.0 ft, then $\tan\theta = 98$ and from Fig. A the ratio of the discharge to the vortex-free discharge is 0.65. If C for purely radial discharge is taken as 0.57 (mean of the Iowa values) the value for our example becomes (0.65) x (0.57) = 0.37, from which

WITH POSEY DISCUSSION OF Stevens & Kolf Paper 1461

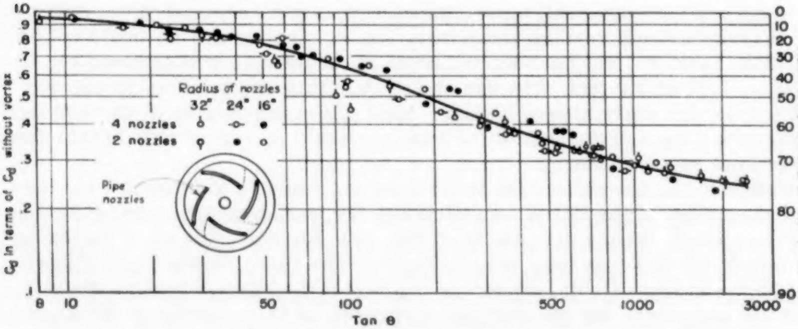


Fig. A Effect of angularity of approach velocity on discharge through bottom orifice in center of circular tank.

Fig. B Stable vortex in the Iowa tank. Part of the inflow is introduced tangentially from pipes pivoted at the bottoms of the four risers.

$$H = \frac{Q^2}{2gC^2A^2} = \frac{1.0}{2g(.37)^2(.196)^2} = 2.94 \text{ ft.}$$

Evidently the trial value of H = 3.0 was a little too large, and a slightly smaller value could be assumed, tan θ recomputed, and H recomputed according to the correspondingly revised value from Fig. A. Such precision would not be justified, however, and the required depth at the periphery of the tank can be taken as 2.95 or 3.0 feet. The use of Fig. A in this way involves an extrapolation from the experimental results. Most of the uncertainty is whether the resistance torque in the tank of the example would be comparable to that used in the Iowa experiments.

Unfortunately the pipes used in the Iowa experiments to introduce the tangential position of the inflow projected into the tank, impeding the circulation to some extent. Hence the results of Fig. A will indicate too much discharge for a tank that has less drag torque than the Iowa tank. Inasmuch as this effect might be appreciable, the writer hoped that the Wisconsin tests would show its magnitude, but the authors' expression of their results in terms of an esoteric variable which can only be evaluated implicitly from laboratory measurements prevents any direct quantitative comparison. Fig. 1 does give some indication that the friction drag is only appreciable near the center, so that conditions around the periphery may not be important.

While attempting to improve the theoretical analysis of the phenomena in order to find a basis for comparing the experimental results, the writer was chagrined to discover that neither investigation included any consideration of what must be a sizable term in the equation—the angular momentum of the effluent. Any future investigation should include measurement of this quantity. Without it (or the more difficult measurement of the torque on the tank) there is no way to evaluate the friction drag.

REFERENCES

4. (as in the original paper)

9. Posey, C. J., Earth's Rotation Has Little Effect on Vortex Motion, Civil Engineering, December 1953, p. 68.

10. Hsu, Hsieh-ching, Vortex Over Outlet, M.S. Thesis, State University of Iowa, 1947 (unpublished).

EFFECT OF LOCAL WEATHER ON AIR POLLUTION PROBLEMS[a]

Discussion by J. Fleming Dias

J. FLEMING DIAS.[1]—Professor Danis' lucid explanation of the climatological factors influencing air pollution problems is commendable. A regional and local study of the weather is in order when dealing with air pollution problems over terrain that is other than flat. There are instances, as pointed out by Professor Danis, where sites even though 20 to 50 miles apart, have considerably different wind velocity distributions. Yet in other cases, sites much further apart, are known to have similar distributions.

The "phenomena" creating air pollution can be justly attributed to the following adverse influences:(1)

1. Meteorological
2. Aerodynamic
3. Terrain

When the regional topography changes rapidly, meteorological influences, and in particular the speed and direction of the wind have to be studied in detail.

An evaluation of the aerodynamic and terrain influences usually involves a wind tunnel study, where the behavior of the "basic plume"(1) can be observed. The basic plume as designated by Sherlock and Lesher is a reference plume in open level country and a stable atmosphere which is relatively free from vertical mixing. Such a plume is easily reproducible in the wind tunnel.

A superposition of the two sets of influences gives the overall performance of the plume at some distance downwind.

The atmosphere has a tremendous capacity to dilute obnoxious gases that are discharged by factories. Depending on local weather conditions the response of the atmosphere changes. If this capacity is to be efficiently utilized, favorable factors such as, (1) stack height, (2) stack gas velocity and (3) stack gas temperature have to be fully explored for maximum and beneficial response of the plume. Due to the complex aerodynamic behavior of the plume, both in the wake and away from the smoke stack, only a systematic wind tunnel study enables one to evaluate independently the relative merits of the favorable factors.

The writer had the privilege of working for Professor R. H. Sherlock, at the University of Michigan, where methods have been developed to combine climatological data and aerodynamic effects to express the overall performance of the gas plume.

---

a. Proc. Paper 1463, December, 1957, by A. L. Danis.

1. Civ. Engr., Goa, Portuguese-India. Research Asst., Univ. of Michigan, Ann Arbor, Mich.

The final results of the wind tunnel tests are expressed in terms of "downwash"(2)—a term that designates a condition in which the bottom of the smoke plume has been driven by aerodynamic forces to descend below some specified height at a specified distance downwind.

It is easily seen that by maintaining any two of the three favorable parameters constant as mentioned above, a valid evaluation of the third could be arrived at. A glance at the adverse influences suggests a study of the variation of the wind velocity for a change in any one of the favorable parameters.

For ease of tunnel manipulation and from proven experimental evidence, the wind tunnel velocity is maintained constant. Instead the stack gas velocity is changed and the downwash is expressed in terms of a velocity ratio "V_r".

$$V_r = \text{Velocity of stack gas/Velocity of the wind}$$

The adverse aerodynamic behavior of the plume depends on the ratio of the momentum of a cubic foot of gas at working temperature to the momentum of a cubic foot of moving air at normal temperature and pressure.

It can be shown that this ratio of momenta is approximately equal to the velocity ratio if the velocities are reduced to a common arbitrary ambient temperature.

Depending on the limit of tolerable downwash there is always a velocity ratio that is critical—the critical velocity ratio (C.V.R.). Time exposure photographs reveal the downwash in terms of the velocity ratio. Fig. 1 shows tunnel photographs and Fig. 2 shows a plot of downwash for several velocity ratios.

It should be noted in Fig. 2 that the region labeled "A B", may be some cases, be rightly called the region of flat response. Any increase in the velocity ratio, which corresponds to a decrease in the wind velocity in the field does not show an appreciable increase in plume height at a specified distance downwind. A case like this, illustrates a condition of partial freedom of the entrapped plume from the turbulent zone, delineated by the "Vortex Sheath".

Any further and appreciable increase in the velocity ratio could very well result in a total escape of the plume. Under actual field conditions the plume would, on escape, be subjected to floatational forces tending to drive the plume upwards. Although the last named effect is not used in the evaluation, it does serve to keep the results on the conservative side.

Results of the aerodynamic behavior as illustrated above and the regional or local climatological data, expressed in graphical form as a probability curve, yield a reasonably good estimate of "the hours of downwash". Hours of downwash expresses the persistence of an undesirable condition of air pollution exceeding the tolerable limit. This unit is commonly expressed in hours per year for a given direction of the compass.

If stack height is the variable then Fig. 3(a) shows a typical set of results. A study of this chart enables one to decide on the optimum stack height consistent with health and economic considerations.

When stack gas velocity is a variable, as is the case under different loading conditions, then the results of the tests could reveal the optimum capacity of precipitators and washers needed to achieve the safe disposal of contaminants into the atmosphere. See Fig. 3(b).

The preceding discussion applies to particulate matter of sufficiently small size to warrant the assumption that its behavior resembles that of a gas.

Besides particulate matter there are some detrimental gases emitted by the stacks. Sulphur dioxide fumes on prolonged oxidation and mixing with

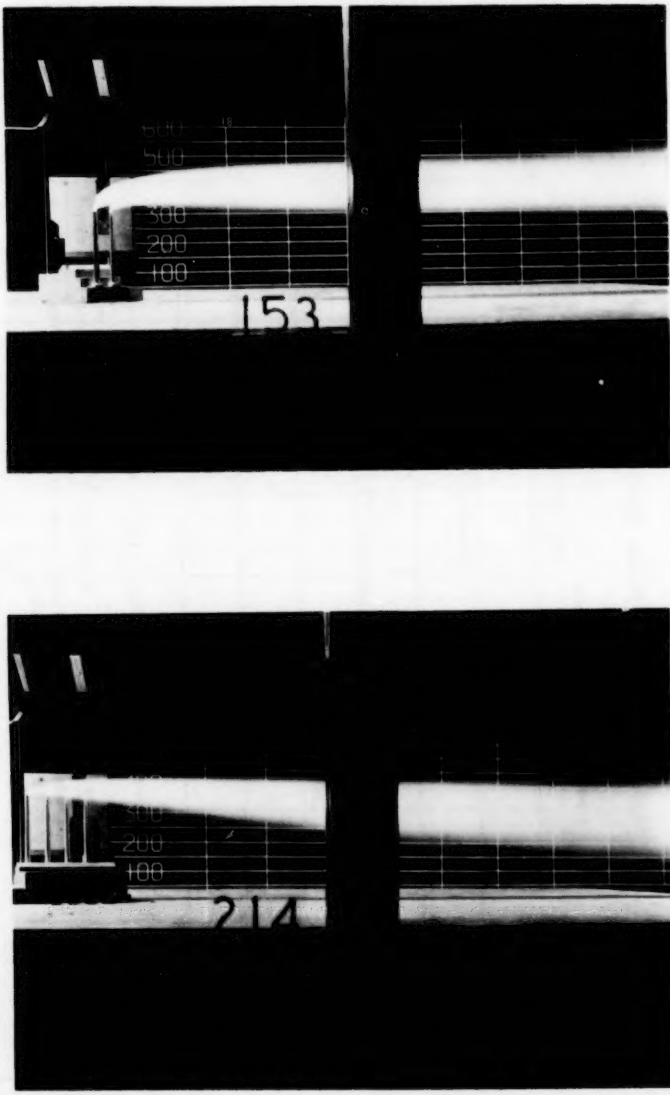


Photo. No. 153

(a) Free flowing plume

(Courtesy of Air Repair)

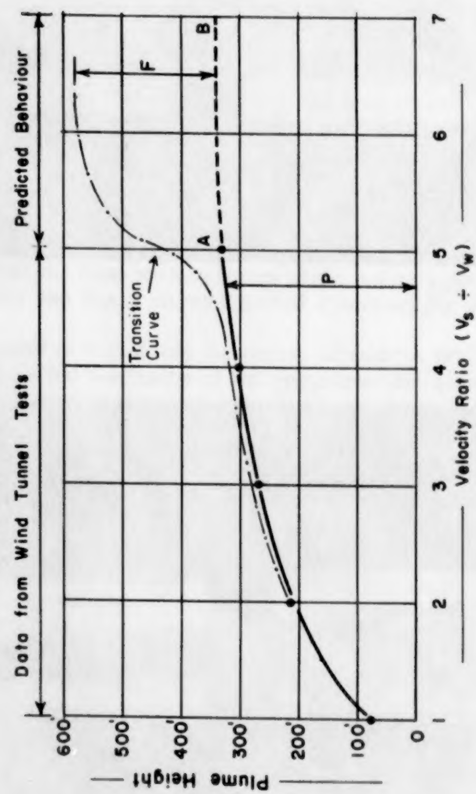
Photo No. 214

(b) Entrapped plume

Figure 1

—— Indicates Plume Behaviour in the Wind Tunnel at ambient Temperature.

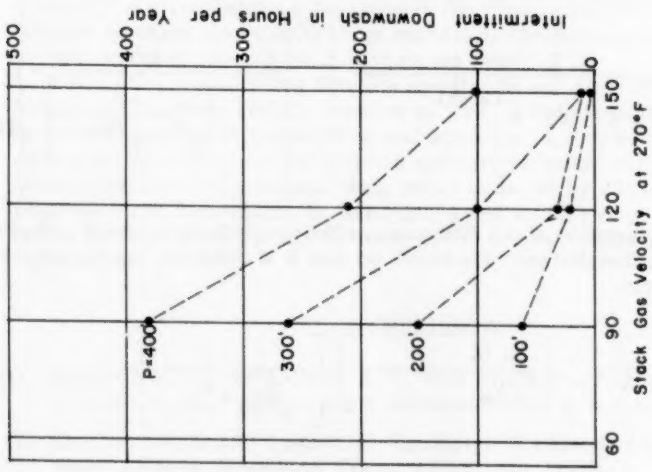
—·— Indicates Expected Plume Behaviour using Hot Gases.



P = Height to Bottom of Basic Plume at a specified distance Downwind

F = Floatational Rise of the Plume when it has Cleared the Turbulent Region Behind the Stacks.

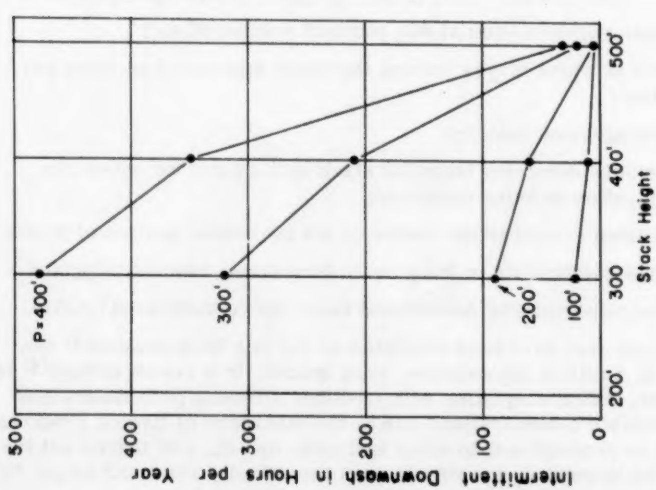
Figure 2



Stack Gas Velocity = Variable

Stack Height = Constant

Figure 3(b)



Stack Height = Variable

Stack Gas Velocity = Constant

Figure 3(a)

water vapor from the atmosphere generate a toxic and corrosive acid $-H_2SO_4$. The control of SO_2 concentrations above certain tolerable limits is essential to prevent toxic damage to human beings and other forms of life.

Sutton's equation is readily available for use in computing the downstream concentration beneath the center line of the plume. Without going into the mechanics of this equation, it may be expressed as follows for sake of clarity:

$$\lambda = \left\{ \frac{2Q}{\pi \, C_z \, C_y \, V_w \, X^{(2-n)}} \right\} \left\{ \frac{1}{e^{\left[\frac{h^2}{C_z^2 \, X^{(2-n)}}\right]}} \right\} \tag{1a}$$

Neglecting the variation of the diffusion coefficients with wind speed and considering a specified distance downwind, so that X is constant, the equation reduces to the form

$$\lambda = \left[\frac{K_1 Q}{V_w}\right] \left[\frac{1}{e^{(K_2 h^2)}}\right] \tag{1b}$$

where

$$K_1 = 2 \div (C_y \, C_z \, X^{(2-n)} \pi)$$

$$K_2 \quad 1 \div (C_z^2 \, X^{(2-n)})$$

λ = the mass concentration of SO_2 per unit volume of air;

Q = the rate at which SO_2 is leaving the stack expressed as mass per unit time;

V_w = the average wind velocity;

X = the distance downwind from the stack to the point for which the concentration is being computed;

h = height from ground to the center of the horizontal portion of plume

C_y, C_z = diffusion coefficients in the y and z directions, respectively; and

n = a diffusion parameter determined from the vertical wind profile.

The coefficients that have been available so far are from studies(3) contributed by M. E. Smith at Brookhaven, Long Island. In a recent article(4) by Smith and Singer, Sutton's equation with modified diffusion parameters has been used to evaluate concentrations due to the discharge of fission products.

Since Q will be proportional to stack exit velocity, Eq. 1(b) brings out the dependence of the downwind concentration on the velocity ratio and height "h" from the ground to the center line of the plume.

The application of this equation to compute relative values of the SO_2 concentrations involves primarily a knowledge of h, and from the corresponding velocity ratio the value of V_w can be computed.

A plot showing the SO_2 concentration at a given distance downwind and directly beneath the center line of the plume is shown in Fig. 4.

From this plot there results an apparent contradiction in regimes 1 and 3. The concentration increases with plume height. From the plotted tunnel plot data it was observed that for greater plume heights, that is for lesser aerodynamic downwash, the velocity ratio is greater. For a given exit velocity of stack gas this represents a decrease in the wind velocity in the field. In Sutton's equation the velocity term appears in the denominator, which accounts for an increased concentration with plume height.

It is highly probable that Sutton's equation is not directly applicable for plume heights under certain regimes of flow. When the plume rises above the turbulence induced by the building and stack and is free to respond to floatational forces, the concentration will actually decrease without any appreciable increase in wind velocity. This effect is shown by a transition curve from regimes 1 and 3 to regime 2. See Figs. 2 and 4. The value of "h" used in Sutton's equation should now be equal to the original value plus the "floatational rise".

REFERENCES

1. Sherlock, R. H. and Lesher, E. J., Role of Chimney Design in Dispersion of Waste Gases. Reprint from Air Repair Vol. 4, No. 2, August 1954.

2. Sherlock, R. H. and Lesher, E. J., Design of Chimneys to Control Downwash of Gases, Reprint of Trans. ASME, Jan., 1955.

3. Smith, M. E. and Singer, I. A., "Relation of Gustiness to Other Meteorological Parameters," J. of Met., Vol. 10, No. 2, April 1, 1953, pp. 121-126.

4. A.I.H.A. Quarterly, Vol. 18, No. 4, Dec., 1957, p. 322.

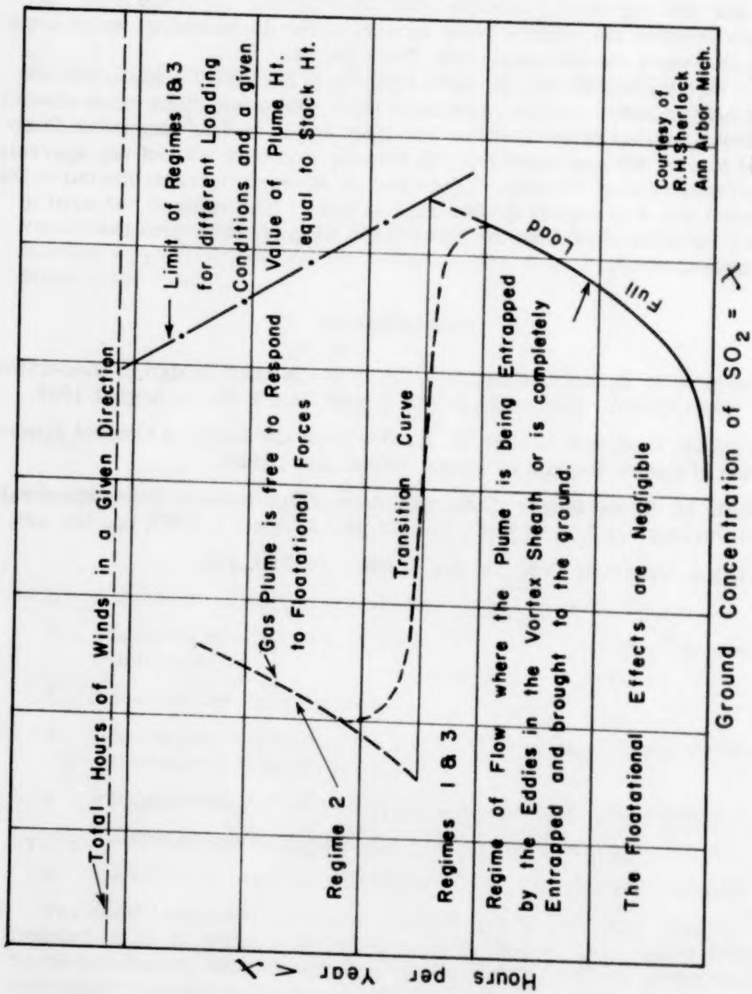

Total Hours of Winds in a Given Direction
Limit of Regimes 1&3
for different Loading
Conditions and a given
Value of Plume Ht.
equal to Stack Ht.
Regime 2
Gas Plume is free to Respond
to Floatational Forces
Transition Curve
Regimes 1 & 3
Regime of Flow where the Plume is being Entrapped
by the Eddies in the Vortex Sheath or is completely
Entrapped and brought to the ground.
Full Load
The Floatational Effects are Negligible
Courtesy of
R.H.Sherlock
Ann Arbor Mich.
Hours per Year < X
Ground Concentration of SO_2 = X

Figure 4

MEDIA CHARACTERISTICS IN WATER FILTRATION[a]

Discussion by H. E. Hudson, Jr.

H. E. HUDSON, JR.,[1] M. ASCE.—Dr. Ghosh is to be complimented on the very thorough work represented by his paper. It appears that there are a few points concerning which further information would be desirable.

The data indicating that head loss through a sand bed is not directly proportional to the bed thickness were obtained by use of filters in which it appears probable that some stratification occurred. While it was attempted to avoid this through the use of very uniform material it is clear that there was some lack of uniformity. It is known that this lack of uniformity will permit partial stratification upon backwashing. The writer's Fig. 1 illustrates such stratification in a bed of 16 x 24 mesh sand in which the particle size determination was made on samples from various depths of the bed by count-and-weigh methods. The data indicate mixing in the middle part of the bed but a concentration of fines at the top. Stratification of this type would produce the sort of data leading to the "depth factor" Dr. Ghosh obtained in flow of both clean and turbid water through the filter.

Study of the data given by Dr. Ghosh on pages 19-21 indicates that this effect was constant for all rates of flow, thus the effect was a characteristic of the experimental beds which probably was due to stratification. The writer found no such depth factor when separate beds of varying depth were tested.

The changes in permeability with time are of extreme importance, and deserve analysis by the method of Hull.(1)

The writer agrees that penetration of sediment into the filter may be deep when coagulants are not used, but would point out that this condition is not at all typical of what happens in rapid sand filtration with the use of chemical coagulants, which produce agglomeration of the particles to such a size that the process of removal is altogether different from the removal of unflocculated Fuller's Earth. Dr. Ghosh reports that the finer filtering materials did produce much greater removal in the first few inches of the bed than coarser materials.

The time rate of clogging as measured by the lost head is an extremely important observation and is directly related to the penetration of material into the bed.

That the final turbidity of the filtrate depends very greatly on the velocity of flow is an important observation. Studies by the writer indicate that incipient turbulence is responsible for the greater penetration caused by higher velocities.(2)

---

a. Proc. Paper 1533, February, 1958, by Dr. Gaurchandra Ghosh.

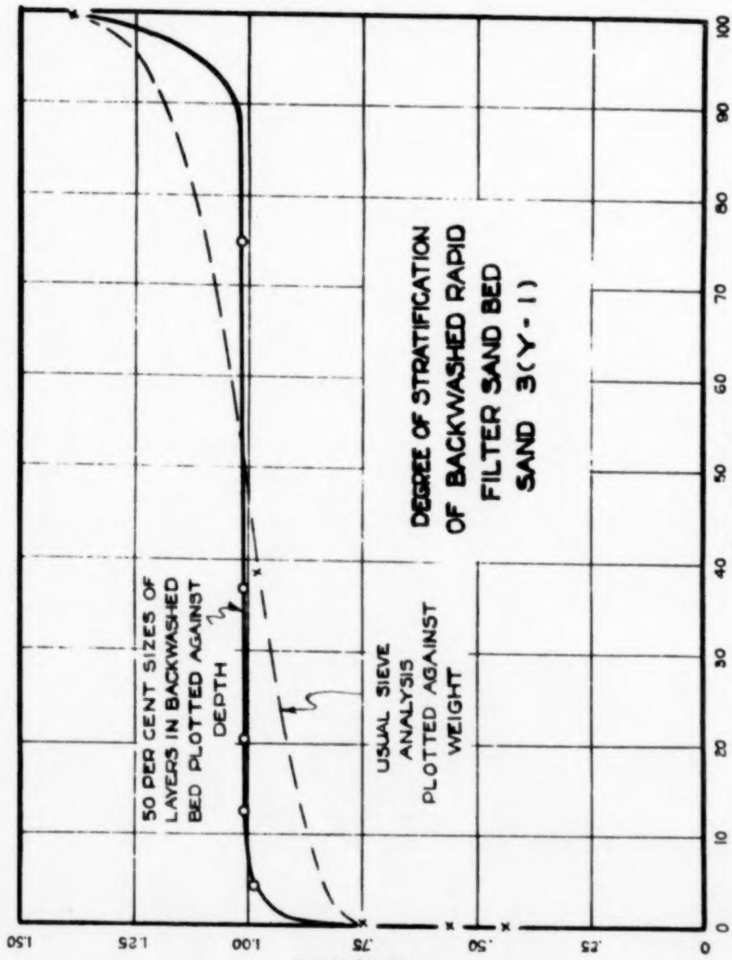
1. Partner, Hazen and Sawyer, Engrs., Detroit, Mich.

It is hoped Dr. Ghosh will be in position to continue his work with beds simulating rapid sand filters treating flocculated water.

REFERENCES

1. Hull, Warren A. "An Analysis of Sand Filtration". Paper No. 1276. Jl. of the Sanitary Engineering Division, ASCE, Vol. 83, No. SA3, June, 1957.

2. Hudson, H. E., Jr. "Factors Affecting Filtration Rates". Jl. A.W.W.A., Vol. 49, No. 9, Sept. 1956, p. 1138.


DEGREE OF STRATIFICATION
OF BACKWASHED RAPID
FILTER SAND BED
SAND 3(Y-1)
50 PER CENT SIZES OF
LAYERS IN BACKWASHED
BED PLOTTED AGAINST
DEPTH
USUAL SIEVE
ANALYSIS
PLOTTED AGAINST
WEIGHT
PER CENT DEPTH
0
10
20
30
40
50
60
70
80
90
100
SIZE IN M.M.
0
.25
.50
.75
1.00
1.25
1.50

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

VOLUME 83 (1957)

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268 (PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277 (HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286 (PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297 (HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306 (SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315 (ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324 (AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

VOLUME 84 (1958)

JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c]

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

---

c. Discussion of several papers, grouped by divisions.